

NOVEMBER 1960



VOL. 52 • NO. 11

Journal

AMERICAN WATER WORKS ASSOCIATION

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Wolman

POLLUTANTS IN WATER SUPPLIES

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DESIGNS FOR PRESTRESSED-CONCRETE PIPE

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FILTER EFFLUENT QUALITY

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US DISINFECTION PRACTICES

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STANDARDS FOR STEEL PIPE

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Journal

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November 1960

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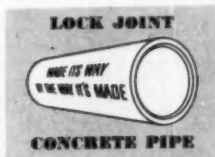
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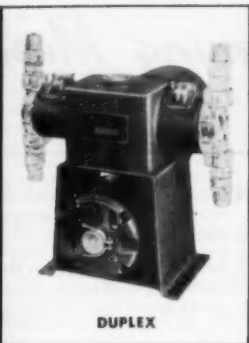
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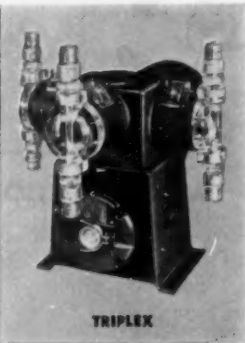
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Coming Meetings

AWWA SECTIONS

Nov. 13-16—Florida Section, at Galt Ocean Mile Hotel, Fort Lauderdale. Secretary, John G. Simmons, Plant Supt., Water Dept., West Palm Beach.

Spring 1961

Feb. 1-3—Indiana Section, at Sheraton-Lincoln Hotel, Indianapolis. Secretary, Chester H. Canham, State Board of Health, 1330 W. Virginia St., Indianapolis.

Mar. 16—New England Section, at Statler-Hilton Hotel, Boston, Mass. Secretary, Ralph M. Soule, San. Engr., State Dept. of Public Health, Boston, Mass.

Mar. 22-24—Illinois Section, at LaSalle Hotel, Chicago. Secretary, Dewey W. Johnson, Research Engr., Cast Iron Pipe Research Assn., 3440 Prudential Plaza, Chicago 1.

Apr. 5-6—West Virginia Section, at West Virginian Hotel, Bluefield. Secretary, Hugh W. Hetzer, Design & Construction Dept., Union Carbide

Chemicals Company, Box 8361, South Charleston.

Apr. 6-8—Montana Section, at Placer Hotel, Helena. Secretary, A. W. Clarkson, Asst. Director, Div. of Environmental Sanitation, State Board of Health, Helena.

Apr. 11-13—New York Section, at Statler-Hilton Hotel, Buffalo. Secretary, Kimball Blanchard, New York Branch Sales Office, Neptune Meter Co., 22-22 Jackson Ave., Long Island City 1.

Apr. 12-14—Kansas Section, at Baker Hotel, Hutchinson. Secretary, Harry W. Badley, Representative, Neptune Meter Co., 119 W. Cloud, Salina.

Apr. 14—California Section, at Biltmore Hotel, Santa Barbara. Secretary, Frank F. Watters, Hydr. Engr., State Public Utilities Com., State Bldg., Civic Center, San Francisco.

Apr. 19-21—Nebraska Section, at Cornhusker Hotel, Lincoln. Secretary, Joseph J. Rossbach, Metropolitan Utilities, 18th & Harney Sts., Omaha.

(Continued on page 8)

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Coming Meetings*(Continued from page 6)*

Apr. 20-22—Arizona Section, at San Marcus Hotel, Chandler. Secretary, A. D. Cox, Jr., Secy. & Comptroller, Arizona Water Co., Box 5347, Phoenix.

Apr. 23-26—Southeastern Section, at Poinsett Hotel, Greenville, S.C. Secretary, N. M. deJarnette, 96 Poplar St., N.W., Atlanta, Ga.

Apr. 26-29—Pacific Northwest Section, at Empress Hotel, Victoria, B.C. Secretary, Fred D. Jones, W. 2108 Maxwell Ave., Spokane, Wash.

Jun. 1-3—Canadian Section, at Prince Edward Hotel, Windsor, Ont. Secretary, A. E. Berry, 72 Grenville St., Toronto, Ont.

Jun. 20-22—Pennsylvania Section, at Galen Hall Hotel, Wernersville. Secretary, L. S. Morgan, 413 First National Bldg., Greensburg.

OTHER ORGANIZATIONS

Nov. 27-Dec. 2—Annual meeting, ASME, Statler Hilton Hotel, New York, N.Y.

Dec. 12-14—National Conference on Water Pollution, Sheraton-Park Hotel, Washington, D.C.

1961

Jan. 29-Feb. 3—AIEE Winter General Meeting, New York, N.Y.

Jan. 29-Feb. 3—ASTM Committee Week, Netherland Hilton Hotel, Cincinnati, Ohio.

Jan. 31-Feb. 1—3rd Sanitary Engineering Conference, University of Illinois, Urbana, Ill., cosponsored by the State Department of Public Health and the university's Department of Civil Engineering. Write: Ben B. Ewing, Assoc. Prof. of San. Eng., Univ. of Illinois, Urbana.

Mar. 13-17—National Assn. of Corrosion Engineers, Statler Hotel, Buffalo, N.Y.

SHORT COURSES

Nov. 14-18—Course on "Radionuclides in Water," R. A. Taft Sanitary Engineering Center, Cincinnati, Ohio. Write: Chief, Training Program, 4676 Columbia Pkwy., Cincinnati 26, Ohio (or to USPHS regional office).

1961

Jan. 9-20—Course on "Orientation in Radiological Health," R. A. Taft Sanitary Engineering Center, Cincinnati, Ohio. Write: Chief, Training Program, 4676 Columbia Pkwy., Cincinnati 26, Ohio (or to USPHS regional office).

Jan. 23-Feb. 3—Course on "Basic Radiological Health," R. A. Taft Sanitary Engineering Center, Cincinnati, Ohio. Write: Chief, Training Program, 4676 Columbia Pkwy., Cincinnati 26, Ohio (or to USPHS regional office).

Feb. 9-10—Short course on "Treatment of Water for Filtration," cosponsored by the University of Michigan and the Michigan Section, AWWA. Write: Director, Continued Education, School of Public Health, Univ. of Michigan, Ann Arbor.



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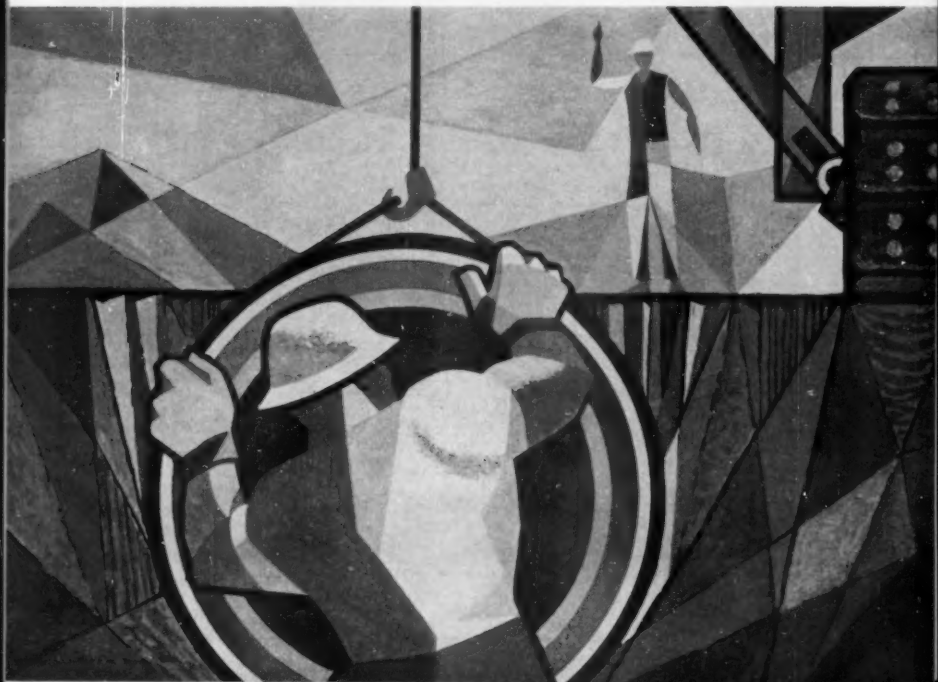
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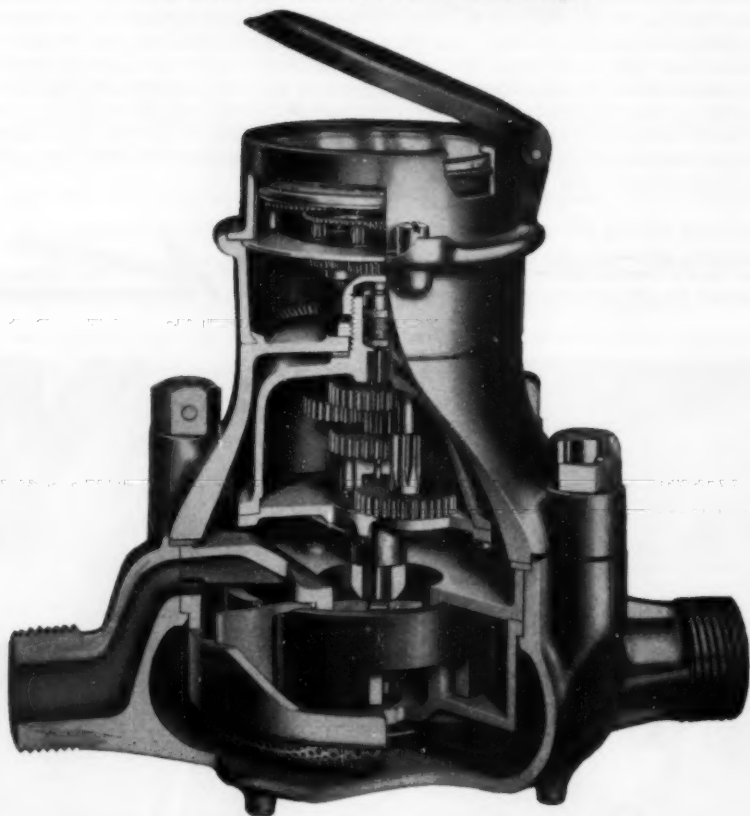
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PIPE AND CONSTRUCTION CO.



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- all-bronze, split case
- perfectly balanced oscillating piston
- exclusive frost-proof bolt design



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CALMET METER DIVISION

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In all phases of water treatment—

In reservoirs, wells, pipe lines, filtration equipment . . . dry, free-flowing HTH with 70% available chlorine kills germs, algae, odors, and fungi fast. Its high rate of stability keeps HTH at original strength longer, too. Comes in granular or tablet form . . . easy to use and store . . . packed in cans, pails or drums. Write for the name of your nearest HTH distributor.

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Chemicals Division • Baltimore 3, Maryland





**RED WATER?
TURBIDITY?**

Eliminate these problems
with GFC Aerators
and Filters

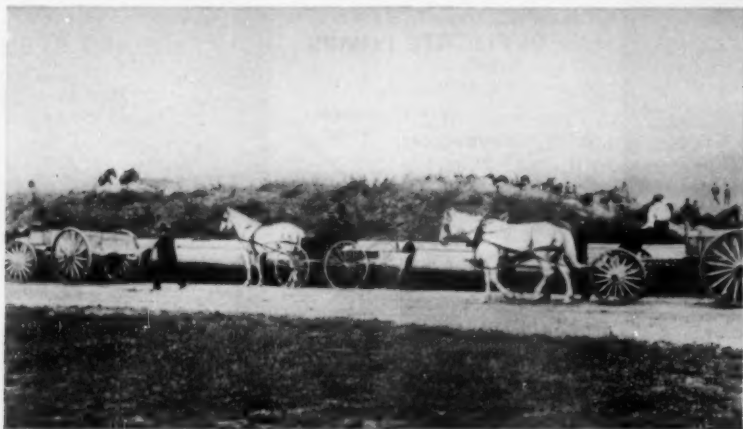
Iron and corrosive, odorous gases cause red water, leaky pipes and clogged meters. Unless corrected, these problems will result in loss of revenue, and consumer complaints.

GFC Forced Draft Aerators and Filters can solve these problems. They are designed for easy assembly, dependability and long life.

Ask about our new aluminum and fiberglass Aerators. Write for your copy of our new Aerator and Filter Plant bulletins with design data and problem analyses.

**GENERAL
FILTER**  **co.**
WATER PROCESS EQUIPMENT AMES, IOWA

AERATORS • FILTERS • SOFTENERS • CLARIFIERS
FLASH MIXERS • FLOCCULATORS • PNEUMATIC CONTROLS
ROTARY DISTRIBUTORS • SLUDGE SCRAPERS



1896-laying New Bedford's 48-in. steel main (and it's still in service!)

The quaint scene shown here occurred during installation of eight miles of 48-in. ID riveted steel pipe in New Bedford, Massachusetts, in 1896. Pipe-laying methods were primitive by present-day standards, but even then steel was a preferred material for transmission mains.

The Water Department reports that there has never been a failure in over sixty years of operation. In fact, they expect the line to remain in service for many years to come.

So it is throughout the country: water departments report steel pipe in service after thirty, forty, fifty or more years. We'll gladly document this statement on request.

Because of today's vastly superior grades of steel, fabricating methods, and coating materials, many engineers conservatively estimate steel pipe's life expectancy as fifty years minimum, and most likely upwards of a century.

Add *long life* to steel's unequalled strength, high flow capacity, and leakproof qualities, and you can see why tar-enameled steel pipe is the logical material for water mains. For more information, contact the Bethlehem sales office nearest you.

BETHLEHEM STEEL COMPANY, BETHLEHEM, PA.

Export Sales: Bethlehem Steel Export Corporation

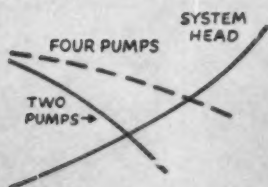


for Strength
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... Versatility

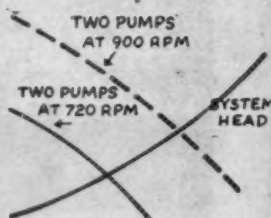
BETHLEHEM STEEL



DUPLICATE PUMPS



HIGHER SPEEDS



Two ways of taking care of future demands for increased capacity are shown here. The chart on the left indicates the performance of 4 pumps

operating in parallel. The chart on the right indicates the same conditions obtained with the use of 2 pumps operating at higher speeds.

DUPLICATE PUMPS OR HIGHER SPEEDS?

A discussion of the economics of multi-speed pumps for water works

WATER WORKS REPORT

from Worthington



Today, most engineers plan for expansion by providing space for duplicate pumps, or pumps that will operate in parallel giving increased capacity as water demand increases.

Another way of taking care of increased capacity is through the use of higher speeds. By providing for multi-speed operation in the future, it may be possible to save thousands of dollars. Spare parts, as well as maintenance costs, can also be reduced.

The charts above show how this can be done. Suppose your water works now uses two 60 hp centrifugal pumps. Sometime in the future you plan to increase capacity

by one-third by adding two more pumps which will operate in parallel. The total cost of the four pumps and motors installed would be approximately \$26,000.

This same increase in capacity, in some instances, can be accomplished by multi-speed pumping. If you initially made the proper provisions, you could later increase the speed of the two pumps from 720 rpm to 900 rpm. This would give you the necessary capacity increase of $33\frac{1}{3}\%$. And the total cost would be only \$21,000, a saving of \$5,000 or approximately 20%.*

It's not necessary to limit your thinking to two-speed operation. Possibly the most economic proposal for your system would be three or four-speed operation. All public water-supply systems are different and pumping equipment is affected by different capacities, suction conditions, and static and friction heads. Therefore, all factors must be known before deciding on the proper approach. This sort of planning, however, combined with the use of multi-speed pumping when appropriate, can save you large amounts of money.

Ask your consulting engineer about the merits of multi-speed pumping. He'll explain to you its many advantages. And for unbiased equipment recommendations, get in touch with Worthington. Worthington makes all types of pumps and drives and a complete line of engines, compressors, and auxiliary equipment.

For a reprint of an engineering discussion of multi-speed pumping for water works, write to Worthington Corp., Section 105-4, Harrison, N. J. In Canada: Worthington (Canada) Ltd., Brantford, Ont.

*Cost figures are estimated and conservative. No cost has been added for additional space required by the extra pumps.

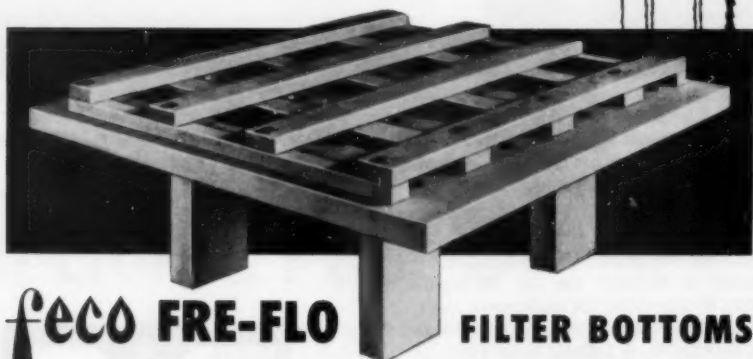


five minutes and a screwdriver

That's all it takes to complete the field assembly of a 24" square unit. This practical design of filter bottom offers superior performance and is the only underdrain system that does not have to be secured to the bottom of the filter. This feature makes it possible to be installed at a fraction of the cost of conventional filter bottoms. Built of strong, non-corrosive materials—*asbestos fibres and Portland cement*—the FECO "FRE-FLO" filter bottom is designed so that ports cannot be blocked by gravel, closed by expansion or enlarged, assuring constant flow and uniform backwash.

The flexibility of the "FRE-FLO" design offers a practical replacement of archaic underdrains which cease to perform adequately.

This false bottom type of underdrain effectively eliminates the velocity head of the backwash water before it enters the gravel layer, making it possible to reduce the amount of gravel.



feco FRE-FLO FILTER BOTTOMS

Write for FREE Literature



FILTRATION EQUIPMENT CORPORATION

271 Hollenbeck St., Rochester 21, N. Y.

Doughnut in the sky

The Graver Aquatore*
— the first new design
in elevated storage
tanks in over a quarter
century—obsoletes
conventional tank design
for service in residen-
tial areas, parks
and wherever else
appearance is
important.



Engineers and public officials can now provide a dramatic, soundly-engineered means of water storage, while eliminating the unsightly tie rods and struts necessary with the usual type of tank. This feat of Graver engineering skill can be built to hold from one-quarter million to three million gallons of water . . . its sleek, steel column can house pumps and other equipment . . . and it costs substantially less to maintain than conventional tanks.

To learn about
this water stor-
age tank that
will be seen
more and more on
the skyline,
write us
today.

In Schererville, Indiana this Graver Aquatore is already a landmark . . . and a tribute to the engineers and officials who specified it.

*Aquatore—Aqua: Latin for water
tore: an engineering term for a
doughnut-shaped object.



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SPEAKING OF PUMPS!

Specifying or buying pumps is a big responsibility. You must be sure the pump you select will operate efficiently, economically and dependably "around-the-clock" or in emergencies.

C. H. Wheeler can help you! We make the type pump you need—centrifugal, mixed or axial flow, both horizontal and vertical—in a wide range of capacities and heads.

Also, we can assist you in designing your pump installation, whether it be for municipal or industrial water supply, sewage, drainage, irrigation, flood control, fire protection, heating, air conditioning, or steam power plant services.

We've been designing and manufacturing quality pumps for almost three quarters of a century, so you know when you specify or buy C. H. Wheeler pumps, you can depend on them. Consult your local representative or write for more information.

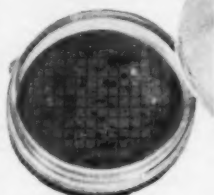
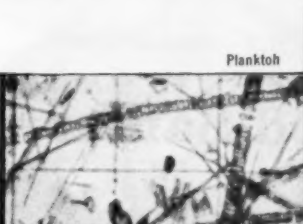
C. H. WHEELER MFG. CO.

19th and Lehigh Avenue, Philadelphia 32, Pa.

Centrifugal, Axial and Mixed Flow Pumps • Steam Condensers • Steam Jet Vacuum Equipment • Marine Auxiliary Machinery • Nuclear Components

Millipore announces a New Laboratory Manual for Bacteriological Water Analysis

Particular emphasis is placed on manipulative details in the enumeration of coliform organisms. Specific techniques are included to isolate plankton algae and determine suspended solids.



The Millipore Procedure for the control of potable waters has been approved as a standard method by:

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Dept. AW, Bedford, Massachusetts

Send me a free copy of the Millipore Manual for Bacteriological Water Analysis.

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The Good Samaritan

We lay no claim to exclusive virtue, nor any patent rights on high moral principles. But we humbly endorse the "Golden Rule" because we have seen the effects of its application to our own business.

Years ago as a struggling new foundry, M & H designed and made a better valve and hydrant. Then, and many times since then, we received most valuable help and encouragement from water works men, engineers and public officials. From the beginning, service-to-the-customer was an M & H watchword. But the reverse—customer-service to the Company—voluntarily and without obligation, surely was a practical application of the "Golden Rule."

These customer "good Samaritans" have stayed with us through the years. They have given us many a lift along the road down which has come today's product. Engineering and foundry research has proved many of their suggestions worthy of adoption—thus helping us to design and manufacture the world's best valve and hydrant.

This customer good-will has been, and is, a human and inspiring phase of M & H steady growth into a position of leadership in the water works industry.

(No. 6 of a Series)

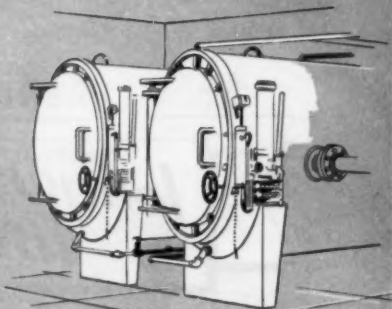
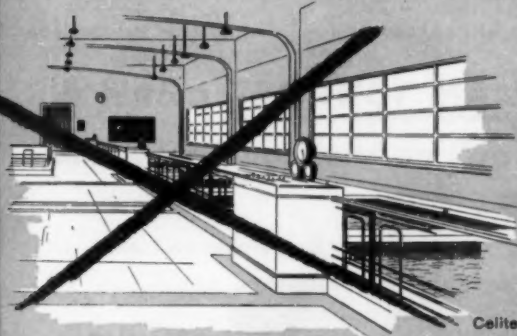


**M & H VALVE
AND FITTINGS COMPANY**

ANNISTON, ALABAMA



1. REDUCE FILTER HOUSING SPACE 75%



Celite diatomite filtration equipment takes up only about one-quarter the space of equivalent-capacity sand filter systems.

2. USE LOWER-COST CONSTRUCTION



Smaller floor space needed for diatomite filtration permits lighter, lower-cost building methods and materials.

You save filtration construction dollars these 2 ways with Celite diatomite

One installation after another proves you need about 75% less filter housing space for equivalent capacity when you use diatomite filtration with Johns-Manville Celite*.

Also, diatomite's lighter, smaller equipment permits you to use lower cost building methods and materials. For example, a simple concrete slab with light framing and siding instead of a large reinforced concrete-and-brick building with built-in concrete filters#.

Diatomite filter equipment is so compact because it provides a much larger filter surface in proportion to size of equipment. In many cases, raw water can be filtered directly—pretreatment facilities can be eliminated. For Celite removes more suspended solids, including amoebae and algae, and turbidity is usually much lower.

Mined by Johns-Manville from the world's largest and purest commercial diatomite deposit, Celite is carefully processed for purity and uniformity. It is available in a wide range of grades for best practical balance of clarity and flow rate with any suitable filter. For further information, see your nearby Celite engineer. For free technical reprints and illustrated brochure, write to Johns-Manville, Box 14, N.Y. 16, N.Y. In Canada, Port Credit, Ont.



*Celite is Johns-Manville's registered trade mark for its diatomaceous silica products.

#See *Comparison Studies of Diatomite and Sand Filtration* by G. R. Bell, Journal American Water Works Association, September, 1956 or write for free reprint.

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*longer
life*

PATENTED SILICONE



STUFFING BOX RACKING

*parts always
available*

*constantly
improved*

WATCH DOG



Split case or frost proof

Offices in principal cities

GAMON METER DIVISION

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NEW JERSEY

A Nalco PRODUCT MANAGER

Answers Questions on Coagulation

**New Chemicals and
New Techniques
Help Solve Problems**



**J. T. Burke, Manager
Nalco Coagulation Chemicals Department**

Question: What are the characteristics of a good coagulation program?

Answer: Continuous production of water of the necessary clarity and quality to meet end-use requirements; with the flexibility to handle changing conditions in input water characteristics. A good coagulation program should also increase the length of filter runs.

Question: What are some of the more recent improvements in the field of coagulation?

Answer: Sodium aluminate is probably the first of the "modern" coagulants. It supplies an alkaline source of alumina with a negative (anionic) charge, and constitutes a coagulant which ideally meets the requirements of principal coagulation process uses.

Natural and synthetic organic polymers are the most recently-developed coagulant chemicals. They are high molecular weight materials which are anionic, cationic, or nonionic; and function as high-efficiency coagulant aids; or both coagulants and coagulant aids. Use of these materials provides greatly increased floc particle size and weight, resulting in increased clarity, longer filter runs, and increased throughput rates.

Question: What is the best approach to solving a specific coagulation problem?

Answer: Take all the factors involved, and accumulate specific in-

formation on them: (a) Make a thorough investigation of the supply water, the equipment employed, the end use of the water. (b) Test coagulation with various chemicals by beaker tests and other laboratory evaluation techniques. (c) Get the advice and assistance of experts as to methods and locations of applying chemical treatments to get maximum coagulation efficiency.

Question: How is Nalco qualified to help solve coagulation problems?

Answer: Nalco's basic background is successful water treatments. The Coagulation Chemicals Department at Nalco is made up of "specialists' specialists": men whose sole job is to provide Nalco Representatives and their customers with the materials and assistance necessary to solve coagulation problems.

No other organization has more experience or better facilities to solve your coagulation problem, promptly and economically. Call or write for action now.

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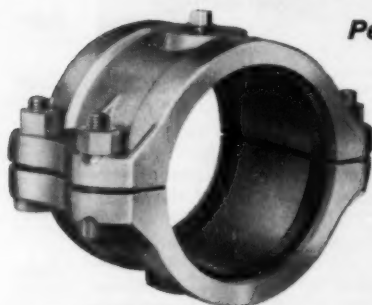
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NEW CLOW

Ductile Iron Split Sleeve



Permits easier, quicker, permanent repair of 4", 6" & 8" transverse cast iron pipe underground breaks

LIGHT WEIGHT Assembled 4" diameter sleeve weighs but 26 lbs. Weight of 6" sleeve is 32 lbs., 38 lbs. for 8" sleeve.

SHORT LENGTH Effective length of 4 inches between end seals on all sizes. Overall length is 6 $\frac{3}{4}$ ".

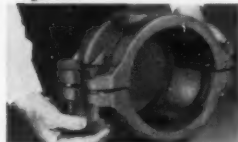
ASSEMBLED IN 3 EASY STEPS



Once the damaged pipe is uncovered, one half of the sleeve is placed on each side of the pipe.



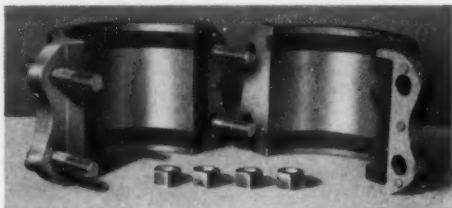
Sleeve halves are drawn together. Pre-assembled rubber gaskets make tight seal.



After the bolts are taken up finger-tight, tightening the four bolts by wrench completes assembly.

TWO-PIECE ASSEMBLY No end glands. Side and end gaskets are pre-assembled and cemented in sleeve grooves.

HIGH STRENGTH All parts are high tensile and corrosion-resistant. Sleeve halves are 70,000 psi tensile ductile iron. Nuts and bolts are high strength, corrosion-resistant alloy.



HERE'S ALL THERE IS TO IT! Illustration shows two-piece glandless construction. Side gaskets overlap ends of circumferential gaskets for tight seal. Four bolts are only accessories. Tapping boss on each half allows maximum tap of 2 inches. Regularly furnished with one sleeve-half tapped for $\frac{1}{4}$ " pipe.

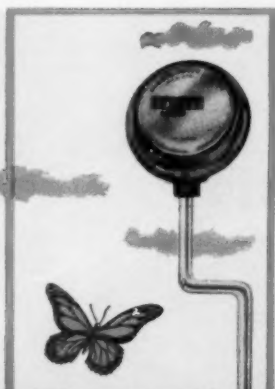
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Iowa Valve Company, Oskaloosa, Iowa

Badger
*Read-o-Matic's**
**cost-cutting story is
big as all outdoors**

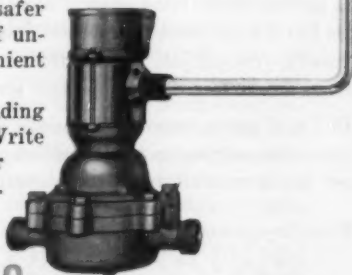


**Pat. applied for in U.S.A.
and foreign countries.*

All over the country water departments cut meter-reading costs and increase efficiency with Badger Read-o-Matic outdoor meter-registers.

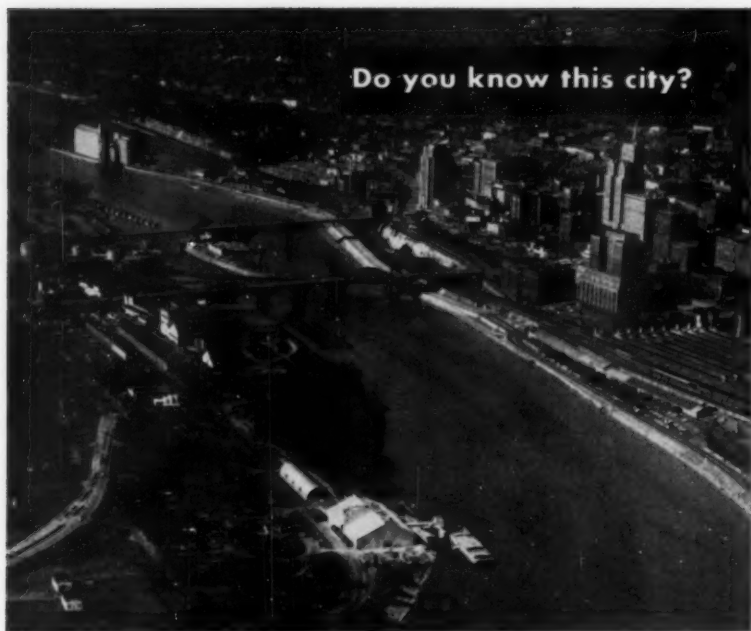
Read-o-Matics can be installed in convenient "reading stations" which a meterman reads quickly and easily. A safer system too, it eliminates dangers of underground vaults and other inconvenient meter locations.

Install this faster, lower-cost reading system in your water department. Write for literature or call your Badger representative for a demonstration.



Badger Meter Mfg. Co.

4545 W. Brown Deer Rd., Milwaukee 23, Wisconsin



DE LAVAL *water works pumps*
provide vital water supply for...

Minneapolis-St. Paul—dynamic twin cities at the headwaters of the Mississippi. Here, as in most major cities of the United States, De Laval water works pumps are doing a year-in, year-out job, proving their reliability in this vitally important service.

De Laval makes water works pumps with capacities ranging up to 100 million gallons per day to meet all water works requirements.



Write for your copies of De Laval Bulletins 1004 and 1005 giving data on these pumps.



DE LAVAL *Centrifugal Pumps*

DE LAVAL STEAM TURBINE COMPANY
822 Nottingham Way, Trenton 2, New Jersey

DL401A



**Spotted on this map are some of
the places that have been served
by the Centriline Process**

Chances are that one of your colleagues has rejuvenated some of his city's piping by Centriline reconditioning. Now here's your chance to find out.

Briefly, this process smoothly lines piping—in place—with cement-mortar, creating a pipe-within-a-pipe. This vastly increases carrying capacity, stops leakage, prevents corrosion and tuberculation, and prolongs pipe life indefinitely. Pressure goes up, pumping cost goes down, and

there's no need to disturb traffic to replace mains. Over 6,000,000 feet of piping, from 4" to 144" diameter, have been treated by Centriline. *Ask your neighbor.*

Write today for a list of nearby officials whose cities have cut inconvenience and costs with this pipe-saving process. Hear what they have to say about Centriline. Then write or call us. We'll be glad to show you how Centriline can help you solve the problems of pipe reconditioning in place.

CENTRILINE CORPORATION

A Subsidiary of Raymond International Inc.

140 CEDAR STREET • NEW YORK 6, N. Y.

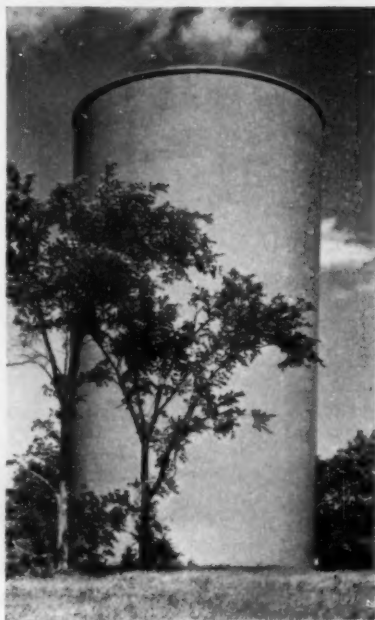
Branch offices in principal cities of the United States, Canada, and Latin America.



EXETER, N. H., has a new STANDPIPE

When New Englanders build, they have long-established habits of building soundly and lastingly. And Exeter, N. H., is no exception. The elm-shaded streets of the town display a succession of handsome colonial homes, sturdily constructed a century and a half ago.

These same habits are strong in the minds of the townspeople today. Thus it was natural for them, when growth dictated a new 1,000,000 gallon standpipe, to specify *steel*.



**... built of STEEL
to AWWA standards!**

STRENGTH
TIGHTNESS
ELASTICITY
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LONG LIFE

... these are inherent qualities of welded *steel* construction. Compare *steel* tanks with other types... you'll see why "steel tanks store water best." You can always specify *steel* tanks with confidence.

**More Than 10 out of 11 Water Storage Tanks Are
Constructed of STEEL**

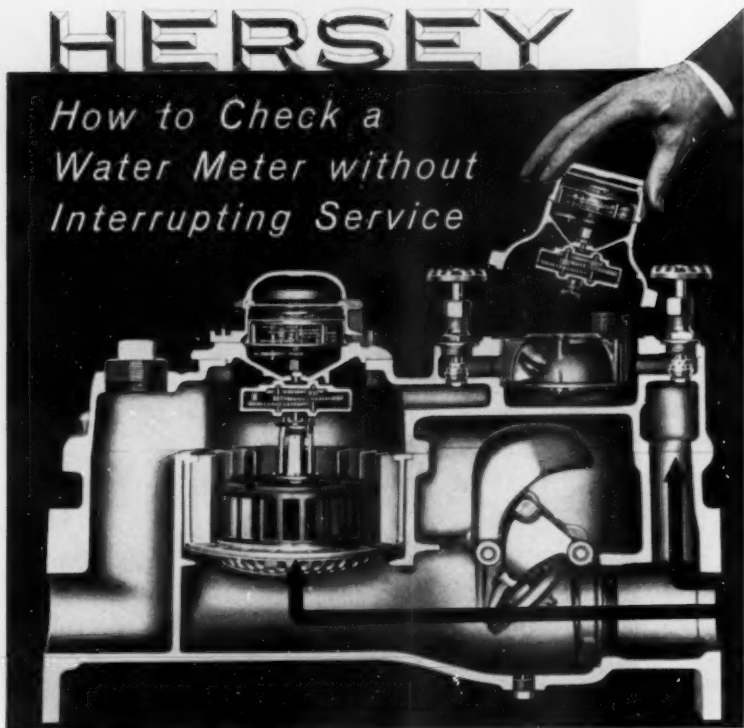
STEEL PLATE FABRICATORS ASSOCIATION

105 West Madison Street • Chicago 2, Illinois

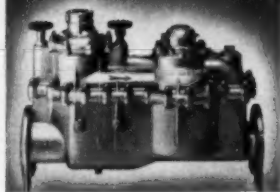


HERSEY

*How to Check a
Water Meter without
Interrupting Service*



HERSEY CT WATER METER *the Leader in its Field*



4" size shown above

That's a Hersey Compound Water Meter having its by-pass meter checked - *without shutting off the main line*. This is one reason these meters are the No. 1 choice when both large and small rates of flow are to be measured. Another reason is the high degree of accuracy obtained by using the extremely sensitive Hersey conical disc type meter on the by-pass.

A special feature of the Hersey Compound Meter is the exclusive swing type lever valve which diverts small flows through the by-pass but, when opened by a large demand, offers no resistance to the flow of water.

These meters are available in sizes from 2" to 10" inclusive (2", 3", 4", 6", all bronze cases—8" and 10" galvanized iron cases). Write for more facts.

Hersey-Sparling
Meter Company

HERSEY PRODUCTS
DEDHAM, MASSACHUSETTS

Branches: Atlanta, Boston, Chicago, Cleveland, Dallas, Denver, Kansas City, Mo., Los Angeles, New York, Philadelphia, Portland, Ore., San Francisco, Seattle.



GOES TO BED WITH ITS WORKING CLOTHES ON

From the moment it is installed, cast iron pipe begins working—will deliver its full flow of water for over a century.

Here in Kansas, as part of a major water expansion program, 15,000 feet of cast iron pipe were put to bed. Ease of assembly, bottle-tight joints, and assured full-flow capacity through cement lining were important factors in the selection of cast iron pipe.

An interesting feature was the uncovering of distribution mains constructed of cast iron pipe manufactured in 1887. The pipe was in excellent condition and was put back in water service.

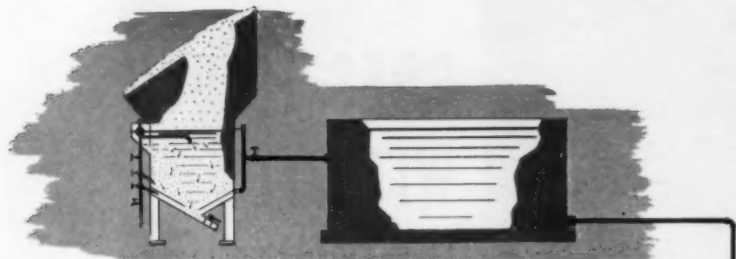
Cast iron pipe is always working. In fact, once it is installed, one can generally anticipate no major repairs or replacements for at least a century.

CAST IRON PIPE RESEARCH ASSOCIATION
Thos. F. Wolfe, Managing Director
3440 Prudential Plaza, Chicago 1, Illinois



CAST IRON PIPE

THE MARK OF THE 100-YEAR PIPE



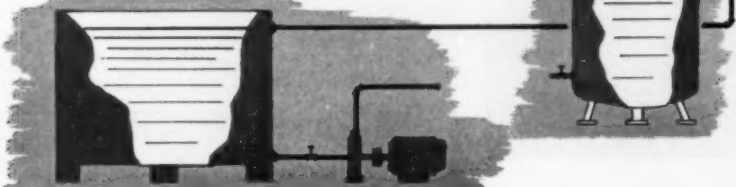
BRINE PIPING

How it can affect design of water softening installations

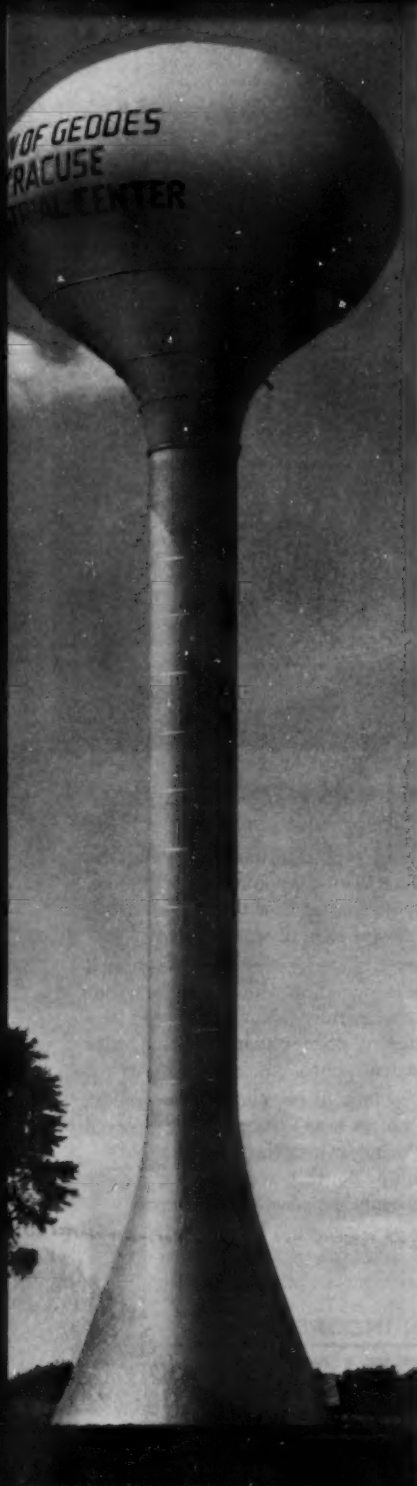
Delivering brine when and where it's needed depends on several important requirements. For instance, the piping must have adequate capacity . . . fittings and valves should resist corrosion . . . and pumps should be correctly located. With the ever-growing amounts of brine called for in today's water softening installations, it's also important to provide for economical expansion of existing piping layouts as needed.

For expert technical assistance on all questions of brine piping, many treatment-plant designers and builders are turning to International Salt Company. 50 years of experience and continuing research in all phases of salt handling and brine production have made International the leading authority in matters concerning salt purchase, storage and dissolving for regenerating ion exchangers. We'd be happy to put our services at your disposal.

Service and research are the extras in STERLING SALT



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why
Geddes, N. Y.

went

14 STORIES HIGH

to assure
better water service
for new industries



**PEDESTAL
SPHEROID
ELEVATED
STEEL TANKS**

This 147-foot PDM Pedestal Spheroid—world's highest tank of its type, so far as is known—brings a 500,000-gallon water capacity into balance with Geddes' expanding supply system. Serving the Syracuse industrial area with assurance of ample water facilities for new industries a-coming, the new PDM unit ties in with a 100,000-gallon tank 3 miles away for effective pressure equalization. Is your community preparing for increasing water demands?

Let a PDM representative work with you, at no obligation.

PITTSBURGH - DES MOINES STEEL CO.

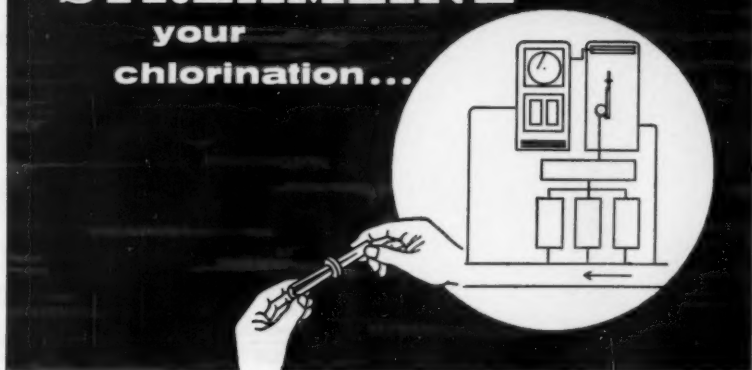
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Concepts of Policy in the Formulation of So-Called Standards of Health and Safety

Abel Wolman

A paper presented on May 24, 1960, before the Joint Congressional Committee on Atomic Energy, Washington, D.C., by Abel Wolman, Prof. of San. Eng., Johns Hopkins Univ., Baltimore, Md.

THE staff of the Joint Congressional Committee on Atomic Energy has posed two primary questions for this discussion. They deal with the exploration of the evolution of standards in the fields of health and safety, other than standards pertaining to radiation control. Such an exploration should reveal precedents, similarities, differences, and difficulties that may make the task of guiding public policy in radiation control more rational or more universally acceptable and intelligible. The questions are paraphrased as follows:

1. For hazards other than radiation, what have been typical practices in public health in developing protection criteria, with particular reference to benefits, risks, and costs in the broadest social and economic sense?

2. How do common arguments applied to radiation hazard appear if applied to historically more familiar hazards?

Fortunately, detailed elaborations of answers to these highly important questions are now available in the superb articles by Parker, Weber, Taylor, Williams, and others in the invaluable joint committee publication of May 1960.¹ This document of approximately 1,250 pages provides for the diligent reader a comprehensive review of the assigned subject. It is too much to expect, however, that the millions of citizens concerned will choose the document for extended perusal. Therefore, brief generalizations will be made here from the wealth of data already in print, with the hope of delineating some historical

peaks and troughs for guidance in the effort to establish standards.

Need for Standards

From its beginning, society, by one means or another, has surrounded itself with restraints. These have had, for the most part, empiric origins—moral, ethical, economic, or spiritual. All the restraints have had the common basis of an assumed benefit to the particular society establishing them. As societies became more complex and more sophisticated, efforts toward both standardization and restraint became more frequent, more necessary, and presumably less empiric, although examples of the last are not as numerous as one might expect.

There are all kinds of standards. Rigid definitions should preclude the loose application of the term "standards" in discussions of standards for radiation control. The procedures often used to establish standards may roughly be classified as:

1. Regularization of techniques of measurement
2. Establishment of limits of concentration or density of biologic life and physical and chemical constituents
3. Regularization of administrative practice
4. Regularization of legislative fiat
5. Specification of materials.

As one reads from No. 1 to No. 5, the procedures become not only more complex but also more nebulous. The hope that authoritative rule will be substituted for thought and judgment is ever present in the minds of some administrators, quantitative-minded precisionists, lawyers, and distressed citizens who are beleaguered by the multiplicity of problems. To resolve these by formula has always been the will-of-the-wisp hope of workers in every

field of human endeavor. It is not surprising that the search for mathematical certainty persists so diligently in the field of radiation, paralleled as it is by the vast uncertainty in the underlying scientific principles that have generated the radiation problem.

Previous Standards and Revisions

The history of the general field of public health practice should disclose precedents, philosophy, implications, and examples of the development of standards. Fortunately, Parker, Weber, and Taylor have discussed many of these aspects.¹ What do they teach us?

The development of criteria for the protection of health has invariably preceded full scientific understanding and acceptance. These criteria have always been subject to reinterpretation, adjustment, and reframing as newer knowledge and experience were forthcoming. Sometimes such revisions have been overdue, even though the machinery for revaluation, both voluntary and official, was at hand. Experience has disclosed that where such criteria have been made rigid by law, revision becomes a heroic effort. Administrative rule is far more flexible.

A familiar example of the revision process is in the changing criteria for drinking water quality. In the last 40 years, several adjustments have been made, with one in the offing for 1960 or 1961. A more striking example of the difficulties confronting the criteria maker is in regard to food additives. With the rapidly increasing number of food additives and the tremendous gap between an understanding of their effects and their hazard to public health, years will undoubtedly elapse before empiricism is separated from scientific accuracy. Even in the assumed under-

standing of the behavior of toxic chemicals, much remains to be learned by the toxicologist, physiologist, and hygienist. The responsible health officer, however, cannot wait for perfect knowledge before interposing barriers between man and industrial poisons. He utilizes the best knowledge at hand, always paying a price for overestimating or underestimating hazard. He is perhaps the prime exponent of those who must act prayerfully in the absence of complete data.

The development of standard techniques of measurement has continued, even though standardization has had a tendency to stultify scientific advance. When the advantages of standardization are weighed against the chaos of dissimilar techniques of measurement, which preceded standard methods, the balance is undoubtedly on the side of a common language.

In the standardization of methods for water and milk analyses, it should not be forgotten that much scientific work, but not all, was at hand. One of the significant provisions in the activity was for continuing revision by carefully established machinery—a frank recognition of progressive science and art. The resulting revisions span half a century and yield valuable lessons for the radiation field, for the voluntary professional societies in the United States had a dominant hand in revising standard methods of water and milk analyses.

Criteria to guide administrative practice have been used with caution in the public health field. Appraisal sheets, coupled with assessments of current practice, have had variable results even when issued by voluntary groups. The fear of making a practice rigid and, at the same time, the desire to facilitate its intelligent administra-

tion are characteristic of attempts to assess current practice. The appraisal sheets have provided aids to improved coverage in public health and have not proved too resistant to new knowledge. Again, relative success with standardization practice is the result, in great part, of the concomitant provision of machinery for the continuing revision of the bases for appraisal.

Calculated Risks

How has the health worker balanced criteria against risk to life? The past record of accomplishment in the field of public health, of which one has reason to be eminently proud, is singularly devoid of such quantitative evaluations. The fact that the American public lives in one of the most protected public health environments in the world is not contested. That this is the result of the composite of public health measures, standard of living, genetic influence, and other factors is likewise true.

But the saving of lives and the extension of life have been the result of public health practice to a significant degree. It must be recognized, however, that the guiding principle that "public health is purchasable" was a qualitative philosophic precept, rarely a quantitative equating of protective criteria against loss of men or dollars. Efforts to apply the latter equation have not been rare, but they have been quite unimpressive in general impact either on the people or on their legislators. Persistent efforts have been made to measure the economic value of the newborn babe, but the western acceptance of the general obligation of society to prevent disease and death is the prevailing one. It is only in the search for criteria for radiation limits that one finds suggestions that it should

be permissible to kill X people to attain Y benefits to society. This has undoubtedly been in the minds of all criteria makers, but rarely has it reached the frank and stark pronouncements of recent years.

Safety Factors for the Public

Has there been a discernible factor of safety in public health criteria invariably in favor of the public? The answer is unquestionably "yes," and the factor of safety has always been large. This principle is well illustrated in drinking water quality standards to protect man against typhoid fever, the dysenteries, infectious hepatitis, and cholera. Obviously the best criteria for detecting a dangerous public water supply are the doctor's certificate showing that the man is dead and the epidemiologic evidence showing that the water he drank killed him. The health official does not wait for such criteria, specific and quantitative as they are. They cannot be applied promptly enough, and they do not provide a wide area of protection. The health official chooses to widen this area immensely by moving to far less specific criteria with broad empiric relationships to disease.

The index preferred for half a century for detecting an unsafe water supply was the coliform organism group—nonspecific, even generally nonpathogenic, and only a qualitative indication of a climate of unsafe quality. But it has served its purpose effectively and is a striking example of intelligent empiricism preceding more refined measures of risk. Fortunately, no easy method of detection of the specific typhoid bacillus was available 30 years ago, because its adoption as a universal indicator would have narrowed measurably the area of safety for the con-

sumer. It is not an unmixed blessing that already the radiation industry is plagued in fixing criteria by a startling multiplicity of specific nuclides and their effects. Another look at the value of gross criteria may be warranted.

The factor of safety was even more enlarged by the essential application of administrative judgment. Water quality appraisal was a composite of an understanding of heredity or origin of source, environmental adjustment or treatment, and of final product. The equilibration of these three factors was a *sine qua non* of assessment and depended on professional proficiency. There were always those, of course, who looked to a single quantitative unit for appraisal. They, in fact, did damage to administrative justice by attempting to oversimplify the complexity of interpretation of many criteria.

Higher Standards

Scientific understanding pressed toward ever more dramatic hopes in public health. Shifts in objective toward even lower death rates became marked as the means for accomplishment became more evident or were created. For example, in the 1920's a residual typhoid fever death rate of 10 per 100,000 was assumed to be inevitable. To attempt to lower the death rate was considered "impracticable." Yet public health measures were persistently enforced, so that the typhoid death rate last year in the United States was one-hundredth of this figure. Was this desire to save lives foolish? It is to be doubted.

Were the results of continually higher standards for health protection unduly costly in dollars? It is rare to have encountered public or private agencies in the past which did not plead poverty or

maintain that costs of correction were excessive. History shows that the public health demand for pasteurized milk was consistently opposed with the argument that capital and maintenance costs would price milk off the market. It is a credit to industry that it meets such challenges while it fights them.

Improvements in water quality have come about rapidly at remarkably low cost, because the technologist has been able to design, construct, and operate plants to meet ever increasing and more rigid criteria. Fear has been expressed that the establishment of too rigid criteria in the field of radiation may stifle progress because of excessive costs of attainment. One may view this fear with some cynicism in the light of the whole history of health and safety endeavor. This fear has always been expressed, but history consistently belies it. Criteria must be based on public health protection and not cost. No one, of course, should advocate excessive and unnecessary restraints. Those restraints most logically suggested, however, within the framework of current scientific understanding should not be resisted solely because resulting costs may threaten to throttle their application.

Standards for Radiation

The lessons of the past in general health and safety practices are easy to read. They are characterized by moving empiric decisions, by persistent reappraisals, by consistently giving the public the benefit of the doubt, by an ever narrowing gap between knowledge and application, by qualitative rather than quantitative slide-rule assessments of hazard, and by objectives calling for the elimination of fatalities

due to disease. The kind of reasoning that proposes as a goal a reduction in the number of deaths due to coronary thrombosis to the level of violent highway deaths that seem to be taken for granted has never been accepted. Such subtleties of philosophic irony, if invoked, would cost many a health officer his job.

The radiation field today is confronted with problems and decisions similar to those in general public health practice, but radiation problems are greatly complicated by the very nature of the biologic effects to be considered. The somatic and genetic resultants are unclear and not fully predictable; perhaps they will not be predictable for many years. Yet one cannot bide one's time in placing restraints on the public and private producer. The latter do not have an unblemished record of self-policing. Hence, society must look to scientific groups and public officials for providing criteria and guides, which are, at times, admittedly uncertain or admittedly tentative. As knowledge increases, reappraisals ensue, either for relaxation or for tightening of criteria. These supposedly fumbling steps have much historical validity and precedent in public health practice. They are unpalatable to the precisionist and to many others seeking to find formulas in place of evolving judgment and declining ignorance.

The day of handbook rule for measuring the hazard of radiation is a long way off. In the meantime, one must act with limited knowledge. In such action, the guiding principle must be the maximum protection of people, not because of sentiment, but because society demands it. An agreed acceptance of a number of consequent disabilities is not an appealing basis for the development, say, of nuclear

power. Industry will do better than rest upon such an affront to man.

Whether the costs of protection will prove to be exorbitant, it is too early to predict. All radiation effort is in evolution; consequently costs are still high. It is not unreasonable to anticipate that all such costs will decline. With this decline, costs of built-in health and safety measures will also be reduced. The past record amply justifies such a prophecy. In the interval during technologic advance, one might profitably follow the wise conclusion of Lauriston Taylor:

We have a deep moral responsibility to make certain that the problem does

not become a critical one for those that follow us. We are thus inescapably compelled to consider, and consider carefully, the question of the long-range uses of all radiation sources whatever, to be certain, first, that any level we set is not seriously exceeded and, secondly, to be certain that no one source causes us to use up our exposure allowance at the expense of other uses, which may in fact be more essential to our overall health and well-being.¹

Reference

1. Selected Materials on Radiation Protection Criteria and Standards: Their Basis and Use. Report of Joint Congressional Committee on Atomic Energy, Washington, D.C. (May 1960).



Effects of Pollutants in Water Supplies

Panel Discussion

A panel discussion presented on May 19, 1960, at the Annual Conference, Bal Harbour, Fla.

Sewage and Industrial Wastes—Ralph E. Fuhrman

A paper presented by Ralph E. Fuhrman, Exec. Secy., Water Pollution Control Federation, Washington, D.C.

THE forms and effects of the pollutants which may enter a water supply with discharges of sewage or industrial wastes are many and varied. Pollutants may be suspended, colloidal, or dissolved, and may be solid, liquid, or gaseous in state. As stated by Klein, in his *Aspects of River Pollution*,¹ solid pollution alone may consist of any kind of solid material, such as sand, gravel, soil, ashes, cinders, clinkers; sweepings from any factory, mine, quarry, or house; any sludge or solid matter from sewage; any vegetable or other garbage; offal or parts of the carcasses of any animal; rubber, paraffin wax, gelatin, straw, paper, or paper pulp; and even bread and butter! A clear means of combating—or even defining—the manifold effects of this heterogeneous array of substances is not readily apparent.

The effects of sewage and industrial wastes on water may be considered under three general classifications: physical, chemical, and biologic. In each of these categories are materials that produce undesirable effects that are undesirable for aesthetic or health reasons. To some extent, natural forces work to rid water of these mate-

rials or change their form so that they do not detract from the water's apparent nor real quality. There is not always time for nature's full cycle of purification to take place, however, and engineers, chemists, and biologists must combine their knowledge to provide artificial means for reaching the same end.

Physical Effects

A sewage discharge into a relatively clear watercourse is immediately apparent unless the dilution afforded is exceptionally large. After mixture in a flowing stream, the color and turbidity are diluted, and whether the effect is slight or one of obvious gross degradation of the water depends mostly on the dilution ratio. It must be remembered that the presence of sewage is much less noticeable if the receiving stream is naturally turbid, although the actual degree of degradation of water quality may be considerable.

With ordinary domestic sewage, the apparent dishwater gray color may be lessened by a small dilution ratio, but in discharges of highly colored industrial wastes, the color may be very

persistent. In a modern dye works some dye vat contents are usually discharged to the plant waste water. Even if the latter is subjected to treatment, it may have only a limited effect on the color itself. Magenta may show a distinct reddish color in a concentration as low as 0.02 ppm. As chemical stability is a required property of a good dye, dilution in the receiving stream may produce the only reduction in the color with time. It is for this reason that such materials are often withheld completely from the receiving stream and given a maximum opportunity for reduction by lagooning or some other similar treatment. Occasionally, wastes may interact to reduce each other's undesirable qualities, but such synergistic dividends must be considered incidental and can rarely be planned.

Increased turbidity is essentially a result of direct dilution immediately below a point of waste water discharge. It is the nature of the material causing the turbidity rather than the existence of turbidity itself that is a serious pollution consideration. Turbidity usually is readily reduced by the self-purification processes in the watercourse. Of course, turbidity caused by very fine particles responds more slowly to both natural and artificial coagulation processes. For example, the very fine rouge which may be discharged from a glass works has a lingering, degrading effect on the water, accented by its color.

Although it is hardly a physical effect, odor does affect the physical sense of smell. The intensity of odor, which is a measure of their potential annoyance, is dependent on the waste products carried by the discharge. The most noisome odor producers in sew-

age include hydrogen sulfide, mercaptans, indol, and skatol. The presence of any of these is indicative of gross pollution indeed, and they are associated with inadequately treated waste water discharges.

The range of odors from industrial waste waters is as extensive as the variety of substances in these waste waters. The many exotic new chemical compounds being created daily produce a virtually limitless assortment of odors and vastly complicate the problem of waste treatment.

The effect of waste water temperature on receiving waters is of much greater significance than might first be believed. Any elevation of temperature is usually detrimental from the water supply standpoint, but in a stream that has the job of assimilating organic matter brought in by waste water, temperature imposes a double burden. First, the rates of biologic activity are increased and, second, the amount of oxygen available for respiration is reduced by the lower solubility of oxygen at higher water temperatures. In addition, the microscopic life which is responsible for much of the stream's recuperation may be affected by changes in temperature or may be destroyed at temperatures well within the reach of natural watercourses receiving heated waters. Temperature changes are being considered more and more as a form of stream pollution in the United States. Although it would occur only under the most unusual circumstances, an increase in water temperature could be a benefit in situations where low temperatures would inhibit biological activity or where freezing would occur. Such instances would be quite unusual, but may be increasingly possible in the future.

Chemical Effects

The chemical effects of minerals discharged with domestic and industrial waste water are many. Acids and alkalis are frequently discharged by the modern industrial plant. Virtually all common acids or alkalis are found in streams, and the effects of very high or very low pH values have been frequently observed. The acid mine water flowing into a stream draining a coal-mining area is a direct example of a large acid discharge. Frequently such streams are so low in pH that all biologic activity ceases until diminution of this effect by dilution or neutralization has been accomplished. Kier liquor and mercerizing solutions for cotton processing, on the other hand, produce waters with pH values of 12-14. It should be noted that municipal waste waters are usually slightly alkaline so that their addition to an acid stream tends to neutralize it. Alkaline discharges of some industrial waste waters have been known to neutralize acid flows and help the recovery of the stream through biologic activity at a nearly neutral pH.

Toxic inorganic substances which may be present in waste waters include free chlorine, ammonia, and hydrogen sulfide, as well as the salts of many heavy metals, such as copper, zinc, lead, nickel, and others. Sufficient concentrations of these substances are toxic to the normal fauna and flora of the watercourse and thus interfere with self-purification. In addition to the interference effects, the dangers of these substances as poisons are obvious. Arsenic and lead are to be watched particularly in this regard. Because of the limitation on chromium imposed by most water pollution con-

trol boards, industrial waste waters that may contain this element should be under constant surveillance.

Dissolved minerals are usually of much less concern, and even public water supplies may contain significant concentrations, depending on the source of water. Biologic activity is tolerant to moderate amounts of these substances, though quantities must be evaluated from the standpoint of the USPHS Drinking Water Standards.² As the concentration of dissolved minerals is generally reduced only by dilution, the discharge of waste waters carrying large quantities must be carefully controlled where water supplies are involved. Because of low cost, brine solutions from an industrial process or from water-softening operations are sometimes discharged with little consideration of the effect on the receiving watercourse. From such practices, the danger of exceeding chloride limitations for a public water supply is far more important than the interference with biological balance in the receiving water.

Inorganic insoluble materials, such as a clay from gravel-washing plants and mixtures of clay and other fillers from paper manufacturing, cause undesirable increases in turbidity. Sometimes, such material may be helpful in the coagulation-sedimentation process, but the additional solids burden must be considered.

Biologic Effects

In the end, it is the biologic effect of polluting substances that is of greatest public concern. If the water has undesirable physical or chemical qualities, the user may be driven to other, apparently safe sources but, quite possibly, with greater danger.

Certainly the newest group of surface water pollutants are radioactive materials. The physiologic effects of these substances are of very great importance. The world has just crossed the threshold of the radiologic era and information on radionuclides is developing rapidly. Limitations of exposure have been established for the various isotopes and work is progressing steadily to determine background radiation levels as an index to the amount of man-made radioactive pollution taking place. In the past, radioactive wastes originated in a relatively small number of installations, but in the future there will be such a multiplicity of sources of these wastes that no major watercourse is likely to escape their intrusion.

To many, the effects of biologic pollution are of transcendent importance. Certainly, with the waterborne outbreaks that still occur in this enlightened world, there would seem to be justification for this stand. The animals and plants that may live in the stream and transmit disease or have other undesirable effects on those using the water comprise an imposing list. In this group may be included pathogenic bacteria, algae, fungi, viruses, protozoa, and parasitic worms. The usual source of such organisms is from domestic sewage rather than industrial waste water, and the used water may form the link in the chain of infection from host to host for these organisms.

The classic waterborne bacterial infections include typhoid, dysentery, and cholera. In the United States, cholera is virtually unknown and typhoid is sufficiently rare that medical students in the vicinity of a hospital that may house a case would be summoned from miles around because of

the rareness of the opportunity to view the symptoms.

The dysenteries (amebic and bacillary) are much more prevalent but less spectacular because fewer deaths occur. It is possible that many minor cases which do not become disabling and therefore are not attended by a doctor go unrecognized as waterborne.

As viruses are universally present in the discharges of patients, influenza, smallpox, poliomyelitis, and yellow fever viruses could be expected in any domestic waste waters where cases of these diseases exist. Of these viruses, poliomyelitis virus has been studied to a greater extent than the others as a possible constituent of a municipal waste water. Although this disease has never been proved to be waterborne, the presence of the virus throughout sewage treatment units and sludges removed from them gives reasons to be watchful for a possible spread of the disease.

The infectious diseases are intimately related to organic wastes of human origin. These organic substances give sewage its objectionable characteristics and provide food for the organisms naturally present. In modern waste water technology, domestic waste waters are collected from individual buildings and conveyed to a point of discharge into a watercourse, usually with some treatment to lessen the burden on the receiving water. The treatment processes, which are physically similar to those of water treatment but are more extensive in biologic application, can greatly reduce the organic load. Whether this organic load is placed on the stream's resources as raw or treated sewage, the rate of satisfaction of the demand for oxygen proceeds at well defined

rates varying with the temperature and the concentration of organic matter. This reaction has been formulated as a unimolecular one, in which 20.6 per cent of the remaining organic matter is oxidized per day (at 20°C). In 5 days, the total reduction is only 68 per cent with normal domestic sewage, and it may be much longer with other waste waters. The need for substantial removals is readily apparent for the preservation of stream cleanliness.

The rates given above for the reduction of the organic compounds in sewage may be considered to be the maximum obtainable. Examples of the initial substances involved are proteins, fats, soaps, waxes, and carbohydrates, all of which decompose under the influence of bacterial action. It is for the respiration of the reducing bacteria that oxygen is required in large amounts. When sufficient oxygen is present, the bacteria convert carbon to carbon dioxide, hydrogen to water, nitrogen to nitrates, sulfur to sulfates, and phosphorus to phosphates. If insufficient oxygen is present, some carbon will be converted to methane, nitrogen will change to ammonia and organic amines, sulfur will partially change to hydrogen sulfide, and phosphorus will form phosphine and organic phosphorus compounds. The materials in industrial waste waters undergo similar changes, but with many more complications.

Conclusion

From the standpoint of safe and satisfactory water supply, the reduction

of stream loads at the point of waste water discharge is of great importance. It affords the maximum protection to the downstream water supply and the population it serves. Of course, economic considerations are important, and it is the role of the sanitary engineer to balance the costs against the benefits. Surely it was in the light of such all-encompassing considerations that Karl Imhoff, the eminent German sanitary engineer, said²:

In densely populated and industrially rich countries the maintenance of river cleanliness through good sewage [and industrial waste] treatment is the best foundation for the safe and economical supply of water to the population and industries.

There is much to be learned from this statement; recognition of its validity will increase with the growing thirst of populations and industries. In the years ahead, there is not only the obvious need of serving more of population and industry, but the most serious need of developing waste water treatment processes of greater effectiveness. Present processes of tertiary treatment include stabilization ponds and even rapid sand filters. Refinements in these as well as new processes are to be hoped for in the future.

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ABS Contamination—Graham Walton

A paper presented by Graham Walton, In Charge, Water Conservation Studies, USPHS, Robert A. Taft San. Eng. Center, Cincinnati, Ohio.

Most household detergents and commercial and industrial cleansers contain the anionic surface-active agent, alkyl benzene sulfonate (ABS). As ABS is not found in natural substances, its presence in water is evidence of contamination by sewage or other man-made wastes.

ABS contamination is frequently detectable by frothing, which has been reported to occur at concentrations as low as 0.6 ppm.¹ Although taste and odor of water have frequently been attributed to ABS contamination, they are not due directly to this chemical. Panel tests² made at the Robert A. Taft Sanitary Engineering Center at Cincinnati showed that the odor of ABS is rarely detectable at concentrations of less than 1,000 ppm and that only very sensitive individuals can taste it in water at concentrations as high as 16 ppm. These tests were made using purified ABS. In similar tests³ with a common household detergent, more than half of a 23-member panel detected an off-taste in water containing 2 ppm syndets or more.

Analytic Procedures

Three methods, or modifications of them, have been used for quantitative determination of ABS in water: the methyl green, methylene blue, and the infrared methods.⁴ As the infrared method eliminates alkyl sulfates and most other interfering substances, it is generally considered to give an unequivocal ABS concentration. This usually ranges from 0.6 to 0.8 of that determined by either the methyl green or methylene blue procedures.

Considerable confusion has been caused by the use of different reference standards. The standard adopted by the Taft Center is a formulation provided by AASGP,* and consists of: 62.4 per cent sodium alkylbenzene-sulfonate, 32.4 per cent sodium sulfate, and 3.4 per cent moisture. This material is said to be quite stable and can be used directly, except that 1.602 unit weights are required for one equivalent weight of ABS.

Unfortunately, some laboratories have used undesignated household detergents of variable and unknown ABS content as a standard of reference. Furthermore, the results are frequently ambiguously reported as concentrations of "detergents" or "syndets" or with reference to some standard other than ABS.

Permissible Limits

Although it is not mentioned in the current USPHS Drinking Water Standards,⁵ the technical subcommittee engaged in the revision of these standards is considering a proposal that the ABS concentration should not exceed 0.5 ppm. This would be a recommended limit based on the apparent effect the presence of ABS has on the aesthetic quality of water.

If the effect of ABS on health of humans were the only concern, a much higher concentration could be permitted. A committee of AASGP has

* Portions of this standard ABS are available on request from Roy W. Peet, Association of American Soap & Glycerine Producers, Inc., 295 Madison Ave., New York 17, N.Y.

recently reported on an extensive review of information on oral toxicity of ABS as determined by both short- and long-term studies.⁶ This committee concluded that "the results all indicate that man and animals can tolerate relatively high levels of ABS in drinking water or food without ill effect." Special long-term tests in which rats

ill effect except for a loss of appetite reported by two individuals.

From the information available, it appears inconceivable that anyone would accept the aesthetic quality of water which contained ABS in concentrations that might be injurious to health.

ABS in Surface Waters

ABS concentrations found in raw or treated surface waters are given in Table 1. Although most of these are data for only a single sample at each location, they are believed to represent normal conditions. Exceptions are those for the Neosho and Marais des Cygnes rivers, which show conditions during or immediately following prolonged droughts.

In general, the ABS concentrations that normally occur in surface waters are not significant. Except for the values reported for the Illinois, Marais des Cygnes, and Neosho rivers, the concentrations recorded are well below those usually considered to affect the aesthetic quality of a water.

Hurwitz and his associates⁶ recently reported a study of the Illinois River. Of particular interest are the data for the 100-mi reach between Morris and Peoria, in which there are no appreciable additions of wastes containing ABS. In this 100-mi, the ABS concentration dropped from 0.9 to 0.5 ppm. In view of the dilution involved, the ABS reduction was about 35 per cent. Whether this was due to biochemical degradation or adsorption is not known.

ABS in Ground Water

Wastes and polluted waters from various sources have been incriminated in ABS contamination of ground water. Included are seepage from

TABLE 1
*ABS Content of Some United States
Surface Waters*

Water Source and Sampling Location	Sampling Date	Apparent ABS Concentration ppm	Reference
Illinois River 10 locations, Summit to Pekin, Ill.	1959	0.5-1.3	9
Marais des Cygnes 4 locations, Ottawa to Osawatimie, Kan.	1953	0.08-1.3*	22
	1954	1.4*	23
Mississippi River Several locations	1959	0.02-0.04†	24
Missouri River Several locations	1959	0.01-0.04†	24
Neosho River 8 locations, Emporia to Oswego, Kan.	1953	0.14-0.24*	22
	1954	0.5-1.7*	23
	1957	0.4-1.8‡	25
Ohio River Cincinnati (downstream)	1955	0.06-0.15§	26
Schuykill River Several locations	1959	0.04-0.08†	24
Lake Erie 3 locations	1959	0.00-0.04	27

* 20 per cent of household detergent concentration.

† Treated waters.

‡ Immediately after prolonged drought.

§ Split sample analyzed by six laboratories by infrared procedure.

consumed a diet containing 0.5 per cent ABS with no significant adverse effect have been reported.⁷ Freeman and his associates⁸ reported a study in which six human volunteers consumed 100 mg of ABS, the equivalent of 2 liters of water containing 50 ppm, daily for 4 months with no apparent

TABLE 2
*Studies of ABS Contamination of Ground Water in Housing Developments**

Location	Distance of Well From Sewage System ft		No. of Wells Sampled	No. of Wells Containing ABS	Max ABS Conc. ppm	No. of Wells Containing Indicated Concentration			Reference
	Range	Mean				≥ 0.1 ppm	≥ 0.5 ppm	≥ 1.0 ppm	
Alabama Huntsville		50 ±	11	6	0.4	6	0	0	12
Connecticut	75-100		5	4	4.4†	4	3	2	15
Florida Dade County	50-75	60	21	6	0.2	6	0	0	12
Kansas‡									28
Massachusetts‡									29
Minnesota Coon Rapids Daily & Hurter Subdivision		50 ±	4	2	0.1	2	0	0	12
Thompson Park		50 ±	7	3	0.4	3	0	0	12
Other		96	22	22	1.2				11
Windward Heights		60 ±	4	4	0.1				12
New Mexico Bernalillo County	20-100	50	20	9	0.8	9	1	0	12
New York Suffolk County Amityville Harbor			31	17					10
Babylon			20	19					10
Brentwood		100	27	2	0.2	2	0	0	13
Center Moriches			65	29					10
Copliague	35-80		186	60	3.3				3
Farmingdale	30-80	80	11	0	<0.1	0	0	0	13
Fire Island§			24	2					10
Islip			47	8					10
Lindhurst			45	5					10
Middle Island	30-100	45	21	0	<0.1	0	0	0	13
N. Lindhurst			54	45	4.5				10
Shirley	60-90	75	6	0	<0.1	0	0	0	13
Smithtown	30-100	70	36	1	0.1	1	0	0	13
W. Islip			161	52					10
New Jersey					9.0				30
Pennsylvania*									
Rhode Island Portsmouth	30-150	50	25	24	5.0	24	19	13	1
Virginia Norfolk		50	36	27	4.1	27	8	6	12
Wisconsin La Crosse			13	10	0.4	10	0	0	12
Other			140	63	10				32

* Each unit serviced by its own well and subsurface sewage disposal system.

† Based on 20 per cent of household detergent concentration.

‡ Several instances of contamination reported.

§ Summer colony; only small amounts of detergents used.

|| 30 per cent of well waters examined contained ABS; reference standard other than AASGP formulation.

* Many underground pools known to contain detergents in some quantity.

cesspools and absorption fields used for disposal of household sewage, oxidation ponds for treatment of municipal sewage, natural and induced infiltration from streams or channels receiving sewage, holding ponds for industrial and commercial wastes, and facilities for waste disposal from commercial laundries.

Metropolitan Fringe Areas

The major ground water contamination problem has resulted from construction of housing developments in which each dwelling is serviced by its individual well and sewage disposal system. The relatively large initial investment required to develop community water and sewerage systems and the problems involved in operation and maintenance of such facilities have led builders to resort to individual services wherever geologic conditions are favorable and zoning laws or regulations permit.

The magnitude of the problem is exemplified by the experiences in Suffolk County, N.Y., and the metropolitan Minneapolis-St. Paul area. Davids and Flynn¹⁰ reported that 35 per cent of 600 well waters analyzed contained ABS. They speculated that there were or soon would be 17,000 ABS-contaminated wells in Suffolk County. Early in 1959, the Minnesota State Department of Health undertook a study¹¹ of the problem in the area surrounding Minneapolis-St. Paul. Current estimates are that 50 per cent of the 54,000 private well waters evaluated to date in that area contain either ABS or abnormal nitrate concentrations.

Available information on studies of ABS contamination of water supplies in housing developments where each unit is serviced by its individual well

and subsurface sewage disposal system is shown in Table 2. Included are data from 30 studies in thirteen states, most of which have been made since 1958. ABS concentrations of at least 0.1 ppm have been found in 357 of the 976 well waters examined. Concentrations of 4 ppm or more have been reported in at least three cases. A few of the more interesting studies are summarized.

Flynn and others³ reported the presence of ABS in waters from 60 of 186 wells checked in Copiague, Town of Babylon, N.Y. Copiague, which covers 72 acres, has a housing density of four and a half dwellings per acre and approximately 90 per cent of the lots are occupied. Most of the houses have been built since World War II, the average age being 7 years. Each dwelling is serviced by its individual well and cesspool. The geologic formation is typical Long Island sand and gravel, with little, if any, covering of topsoil. Depth to the static water table ranges 7.5–12 ft. The wells vary 14–56 ft in depth and are located 35–80 ft from the nearest cesspool. ABS contents of 60 well waters ranged 0.1–2.5 ppm, as sodium lauryl sulfate. (A sample of water from one of these wells was later submitted to the Taft Center. The analysis showed 5.0 ppm ABS by methyl green and 3.3 ppm by infrared procedures.) Waters from several wells were analyzed twice, once during November 1957 and again in March 1958. In each case, the later analysis showed equal or greater ABS content. The data indicate that there were no significant differences in the ABS contamination of wells located 35 and 60 ft from the nearest cesspool, but that there was less contamination in the deep wells.

Deluty¹ studied ABS contamination of wells in the Quaker Hill subdivision near Portsmouth, R.I., which consists of approximately 50 homes built between 1953 and 1958. The average lot is about 9,000 sq ft, which, after allowance for streets, would give a

Pennsylvania sandstone, shales, and conglomerates. Water occurs in openings along the bedding plains and in the zone of fractures. The wells which were sampled are drilled to depths of 60-300 ft, the median depth being 145 ft. They are located 30-150 ft

TABLE 3
Reported Instances of ABS Contamination of Ground Water by Other
Than Underground Sewage Disposal

Location	Date	Type of Well	Probable Source of Contamination	Apparent ABS Conc. ppm	Approximate Distance of Travel ft	Reference
<i>California</i>	1959	private	sewage and industrial waste discharged to dry stream bed	0.6		
<i>Colorado</i> Derby Commercetown	1957 1959	municipal industrial	tractor repair garage lagoon induced infiltration of sewage contaminated water	0.6 *	1,000 1,000	33
<i>Connecticut</i> Plainville	1958	municipal	industrial waste, holding pond	0.4-4.0† 0.3-2.0† 0.1-0.9†	300 500 700	15, 34
<i>Other</i>			induced infiltration from contaminated stream	0.6	800	15
<i>Florida</i> Diana	1959	industrial	subsurface disposal of treated laundry wastes			35
<i>Other</i>	1958	irrigation	subsurface disposal of treated laundry waste	10	60	35
<i>Illinois</i> Peoria	1959	test well industrial	recharge of contaminated river water	0.7 0.2	200 1,800	17
<i>Kansas</i> El Dorado Augusta	1956	standby municipal wells	induced infiltration of contaminated river water	2.6		36
<i>Nebraska</i> Kearny	1954-55	private	sewage oxidation pond	2.0† 1.5† 0.3†	800 1,000 4,000	16
<i>Washington</i> Tieton	1958	private	sewage oxidation pond	0.2	300	37
<i>Wisconsin</i> Lac Du Flambeau	1959	private	holding pond for laundry wastes	*	100	38

* Foaming indicated presence of ABS.

housing density of approximately four dwellings per acre. Each house is serviced by its individual well and septic tank with an absorption field. The geologic formation consists of 8-10 ft of overburden underlain by

from the nearest absorption field. The median distance is 60 ft. Of the 25 well waters sampled, 24 contained ABS. The concentrations as determined by the methylene blue procedure ranged from 0.15 to 5.0 ppm ABS.

Coliform bacteria were detected in waters from six wells, including the one free from ABS contamination.

Kiser¹² reported the results of a reconnaissance survey made for FHA. Shallow wells from housing developments in six widely separated areas were sampled. The general criteria used in selecting an area were that the subsurface materials be permeable to a depth of at least 50 ft, the housing development be at least 3 years old, and each house be serviced by an individual well and sewage disposal system. The communities studied were in the metropolitan areas of Huntsville, Ala.; Miami, Fla.; Minneapolis, Minn.; Albuquerque, N.M.; Norfolk, Va.; and La Crosse, Wis. A total of 131 wells were sampled. In each area at least 10 per cent of the waters contained an ABS concentration of at least 0.2 ppm. The maximum concentration found in each of the six areas varied from 0.4 to 4.1 ppm. In one housing development, all nine wells sampled contained at least 0.1 ppm and four contained more than 1.0 ppm.

Lauman and his associates¹³ made a special study of ABS contamination of wells in housing developments where the static water table was at least 40 ft below the surface of the ground. Only three of 101 well waters analyzed contained ABS in excess of 0.1 ppm. The maximum concentration reported was 0.25 ppm. It thus appears that the depth to which waste water percolates through unsaturated sands may be a significant factor, in that it at least delays the time at which a well may become contaminated.

Other Sources

Table 3 summarizes reported incidents in which well water supplies were contaminated with ABS from

sources other than cesspools or subsurface absorption fields. Included are three public well water supplies.

Flynn and others⁸ mention the appearance of syndets in test wells as deep as 95 ft and at distances of 1,000 ft from the source¹³ at Mastic, N.Y., where the vertical and horizontal course of ABS contamination originating from wastes of a self-service laundry is being studied. Samples from this project submitted to the Taft Center showed ABS concentrations ranging from a trace to 13.6 ppm by the methyl green procedure.

Seepage from holding ponds receiving wastes from a plant manufacturing ball bearings caused ABS contamination of a well belonging to the Plainsville (Conn.) Water Co.¹⁵ The industry used about 700 lb of a cleaning compound, containing approximately 42 lb of anionic surfactant, per month. It took 6-7 months for the contamination to travel 500 ft to pollute a 10-in. well. Corrective measures, which included pumping the contaminated well water to waste and a change in cleaning compound, prevented appreciable contamination of a second well 150 ft beyond the 10-in. well. Analyses of waters from the holding pond and 10-in. well showed concentrations of approximately 44 and 2.6 ppm, respectively.

Neel and Hopkins¹⁶ noted that seepage from a sewage oxidation pond traveled underground a distance of $\frac{3}{4}$ mi in 14 months to contaminate water in an irrigation well. The surfactant concentration was 0.3 ppm. They also noted 2.0 ppm surfactant in water from a windmill well 800 ft distant, and 1.5 ppm in water from a domestic well 1,000 ft distant.

A 24-hr composite of Illinois River water collected from the intake to the

recharge pits of the Illinois State Water Survey at Peoria, Ill., was analyzed by the Taft Center. The ABS content was 0.7 ppm. Grab samples from a test well 200 ft from the recharge pit and an industrial well 1,800 ft away showed 0.7 and 0.17 ppm ABS, respectively.

Taft Center Research

Studies, under the supervision of Gordon G. Robeck, are now underway at the Taft Center to investigate the fate of ABS and coliform bacteria and the influence of one upon the other in ground water. Simulated water-saturated aquifers, consisting of 10-ft columns of sand 6 in. in diameter, are used. Water containing the selected contaminants is fed into the bottom at a rate producing a velocity through the sand of about $\frac{1}{2}$ ft per day.

Limited data are available from tests using two different sands obtained from Chillicothe and Newtown, Ohio. The waterborne ABS passed through the Chillicothe sand without appreciable change in concentration, but a substantial reduction in the ABS content of the water occurred when it was passed through the Newtown sand. This apparent removal may result from biochemical degradation, physical adsorption, or chemical reaction with impurities in this sand. Supplementary studies using ABS containing radioactive sulfur will be undertaken in an attempt to explain this phenomenon.

Coliform bacteria have progressed slowly in both sands. Present data are insufficient to determine whether ABS has had an appreciable effect on the distance coliform bacteria penetrate water-saturated sand.

Plans for future studies include an investigation of the fate of viruses under various conditions in water-

saturated sands and the movement of various contaminants in water percolating through unsaturated sands.

Bacteriologic investigations made in the Taft Center laboratories¹⁸ have shown that ABS does not interfere with or inhibit the growth of coliform bacteria and does not present any problem in their detection. The experimental evidence suggests that coliform bacteria are not capable of utilizing the carbon in ABS in their metabolism.

Conclusions

There is adequate evidence that ABS contamination is more widespread than shown by published information. There appears to be a potential problem of ABS contamination wherever wastes containing anionic surfactants are permitted to seep into an aquifer from which water is drawn.

The finding of ABS in water is evidence of the presence of sewage or waste waters. Water having appreciable concentrations of this chemical may foam or contain waste materials producing an undesirable taste or odor. Available information indicates that the aesthetic quality of the water may be affected when ABS levels reach some point in the range 0.5–1.0 ppm.

ABS contamination of surface water should not exceed that concentration which may affect the aesthetic quality of a treated water. As the treatment provided at most water plants is relatively ineffective in removing ABS, its concentration in raw water should not be more than 0.5 ppm. Below this concentration ABS may still be of concern as an indicator of sewage and waste pollution and the accompanying water treatment problems.

The presence even of low concentrations of ABS in private well waters is

a more serious matter. Here the only treatment is the natural cleansing the water undergoes as it percolates through the ground. This treatment cannot be stepped up to overcome increased pollution. Examinations of such well waters for bacteriologic quality are generally made at infrequent intervals. Consideration must also be given to the source of the ABS and to the probability that other contaminants from that source may gain entrance into the well water. In such cases, interpretations of the significance of low-level contamination, such as 0.2-0.5 ppm ABS, should be left to those health authorities having knowledge of local conditions. It should be recognized that ABS is only one of many organic chemical contaminants occurring in both surface and ground waters. Current emphasis on this contaminant is due to its almost universal presence in waste waters, the layman's ability to detect its presence in waters in concentrations of approximately 1 ppm or more, and the existence of reasonably satisfactory laboratory procedures for its quantitative measurement.

Chemical contamination of ground water is far more serious than similar pollution of surface water. Underground waters move but a few feet per day. Years may pass before the contamination is detected, but once it has occurred it may require an equal number of years before the water can be freed from contamination. An extreme example is the Montebello incident,¹⁰⁻²¹ in which wastes discharged from a month's operation of a chemical manufacturing plant reportedly contaminated a municipal water supply for 4-5 years. It appears essential that steps be taken to protect ground water resources from chemical contamination rather than to attempt cor-

rective measures after contamination has occurred.

Acknowledgment

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Taste and Odor—Herbert O. Hartung

A paper presented by Herbert O. Hartung, Exec. Vice-Pres., St. Louis County Water Co., University City, Mo.

Not all tastes and odors in public water supply sources are caused by pollutants. Some tastes and odors are from natural sources and were present in some United States streams even before man began using these streams for disposal of liquid wastes. These were from decaying vegetation, algae, microscopic organisms, and natural minerals. The most troublesome and objectionable of tastes and odors in modern public water supplies, however, are caused by materials carried into the streams by sewers and by waste from industrial and agricultural activities.

Problem of Pollution

The threshold odor numbers of sewerage wastes emptying into streams have been measured to be between 500 and 12,500. This means that if a metropolitan area of 750,000 people discharges 75 mgd (100 gpcd) of sewage having an average threshold odor of 6,500, 750,000 cfs of river flow would be required to dilute the sewage enough to reduce the odor to a barely perceptible level. This flow is more than fifteen times the average flow of the Missouri River at St. Louis. Fortunately, rivers like the Missouri River can assimilate some taste- and odor-producing substances through biologic degradation. This means that the dilution requirements of sewage, for taste and odor control, are somewhat lower than those cited above. It is readily apparent, however, that a single metropolitan area can severely damage a river for downstream water users.

Pollution from oil refineries and from upstream pollution producers are particularly troublesome causes of taste and odor. The intensity of tastes and odors caused by petroleum wastes has been illustrated by several experiences of petroleum pipeline breaks at river crossings. For example, a petroleum pipeline break at a Missouri River crossing in St. Louis County caused such severe taste and odor problems that the nearby water plant intakes were stopped until system storages were depleted as a necessary measure to protect water consumers. Those tastes and odors could not have been removed by the existing water treatment plants. Also, oil slicks reported by the Corps of Engineers downstream from Kansas City, Mo., have, on numerous occasions, caused very difficult taste and odor removal problems at both the St. Louis and St. Louis County water treatment plants.

Industrial wastes from paint and varnish manufacturers are also very troublesome taste and odor producers. Periodic discharges from paint and varnish mixing plants upstream from a water plant have been reported to be the cause of very objectionable taste and odor, which could not be removed by free residual chlorination and normal amounts of activated carbon.

It would be very difficult to list all of the pollutants that cause taste and odors in streams. The list of such materials is too extensive and varied. Such a list would include almost every industry. In addition, mixtures of pollutants that are odorless in them-

selves also sometimes result in odorous compounds. Other pollutants, when chlorinated, very often become the cause of tastes and odors in public water supplies. These pollutants are particularly troublesome to the water treatment plant operator, because the tastes and odors resulting from chlorination of the pollutants occur at a point in the water treatment process where it is too late for corrective treatment.

Carbon Filter Extraction

Although it is impossible to itemize all substances causing tastes and odor in water, much of such organic pollution is being measured by the chloroform-soluble carbon filter extract (CSCFE) procedure. Most of the organic content measured by this method is considered to be from man-made wastes, because only relatively small amounts of chloroform-soluble organic compounds are found in unpolluted lakes and streams as compared to the amounts found in known polluted waters.

There is a limited correlation between tastes and odors and the amount of organic compounds in water. Waters free of objectionable tastes and odors in public water supplies have consistently had less than 200 ppb of chloroform-soluble organic compounds; however, some waters have had very objectionable odors even though the chloroform-soluble organic content was less than 100 ppb.

When the CSCFE organic compounds are fractionated into six basic groups, the odorous materials are mostly in the neutral and weakly acid groups. These two groups still comprise a very large number of specific organic substances, and must be further characterized before specific ma-

terial types causing tastes and odors can be identified.

Missouri River Measurements

Even though further development in methods is necessary, the routine measurement of the total CSCFE can be a very useful indicator of the organic pollution carried by streams and of the potential for taste and odor problems. Such routine measurements are being made by the water plants along the Missouri River. It is interesting to observe that the average amount of these organics in the Missouri River at Yankton, S.D., in 1958-59 equaled 4,000 lb/day. At St. Louis, however, 800 mi downstream from Yankton, the average amount of these organics was 17,000 lb/day. This increase reflects the pollution contributed to the river between Yankton and St. Louis. In addition, the difference in the composition of the organics at Yankton and St. Louis is especially noteworthy. For example, at Yankton, 720 ppb of chloroform-soluble organic compounds was required to produce a threshold odor, whereas at St. Louis only one-third as much has been required to produce an odor.

The ability of streams to remove some organic material by absorption, sedimentation, and biologic degradation is well recognized. A considerable amount of the CSCFE from polluted waters has been found to be very resistant to biologic degradation, however. Orthonitrochlorobenzene, for example, a cumulative toxic poison dumped into the Mississippi River at St. Louis, was found virtually unchanged in the river at New Orleans. The inability of streams to assimilate many industrial wastes is causing progressively more troublesome taste and

odor problems and other reasons for concern in downstream water treatment plants. Significantly, Missouri River taste and odor problems are much more frequent and troublesome at St. Louis than at Omaha or Kansas City.

The CSCFE test is not suitable for day-to-day water plant control. The sampling usually extends over a 2-week period and extractions require an additional 2 days. Thus, one of the most pressing problems before the water supply industry is the development of further techniques for the identification and measurement of tastes and odors and the substances which cause them. Such laboratory procedures and techniques are very necessary for water quality control, for pollution abatement enforcement, and for the development of a truly meaningful water quality standard. The best available control test for tastes and odors is the threshold odor test, which is more qualitative than quantitative. The obvious weakness of this test is that tastes and odors perceptible or objectionable to one person may be unnoticed or unobjectionable to another. Without a good analytic laboratory tool for measuring and monitoring tastes and odors, water treatment plant control of the problem is extremely difficult.

Tastes and odors are not always continuously noticeable in river waters; the Missouri River affords an example. Flow in the river, the strength of wastes dumped into the river, variations in biologic activity brought about by temperature changes and other factors all vary from day to day. As a result, the occurrence of tastes and odors is periodic and relatively unpredictable. Thus, a good detection test is needed for tastes and odors in

raw water, as well as for those substances in the raw water which result in tastes and odors after chlorination. In addition, raw-water sampling and testing must be frequent if tastes and odors are to be kept from the water consumer.

St. Louis County Controls

In the absence of a good detection test, the St. Louis County Water Co. employs several controls. First, treatment plant basin operators and laboratory personnel frequently (every few hours) observe for changes in odor coming from the water as it spills over the presedimentation basin weirs. Because of the large mass of water spilling over the weir, this in-plant odor detection method is often a more sensitive odor indication than the laboratory threshold odor test. Second, when a change from the normal threshold odor number of 2 to 4 is noted, tasting of the water in all of the treatment basins is done more frequently. Third, basin water samples, following prechlorination, are discriminately tasted by the laboratory staff several times each day. Fourth, an increase in chlorine demand beyond the winter normal of 4-5 ppm is regarded as indicative of increased pollution and possible taste and odors. Fifth, reports of taste and odors are regularly received from upstream water plants through a cooperative arrangement.

To date, a majority of the tastes and odors in the Missouri River can be removed by the addition of 8-12 ppm chlorine with approximately 6 hr contact time. For some tastes and odors, 5 ppm or more of activated carbon has been required after oxidation with chlorine. In a few instances, water consumers have even complained that

this treatment was not adequate, however.

Taste and Odor and Public Health

During one period of taste and odors in the Missouri River, the Missouri Division of Health made a cursory survey and showed a seeming correlation between tastes and odors and outbreaks of intestinal enteritis. Some available information justifies the speculation that the presence of man-made pollution producing taste and odor should be considered an important public health hazard and not merely a threat to the aesthetic quality of the water. For this and other reasons, the water supply profession has a responsibility to strive industriously for the abatement of pollution in public water supply sources: The water supply profession should accept the responsibility to study and devise new, improved methods both for waste treatment and water treatment to the extent necessary to guarantee absolute protection of the water consumer. Organic compounds of the type being measured by the CSCFE method are not completely removed in conventional water treatment plants, or, for that matter, in most sewage treatment plants. Many of these compounds are unaffected by biologic-

oxidation treatment, nor can they be oxidized by chlorination. In addition, economically feasible waste water treatment and drinking water treatment methods have not been developed to the extent that there can be indiscriminate dumping of wastes into sewers and rivers.

In spite of these facts, too many pollution abatement programs require the construction only of primary waste treatment plants. Obviously, the water utility operators should present their problems to those concerned with pollution abatement.

Conclusion

Taste and odor removal and pollution abatement are socio-economic problems as well as technical problems. It must be determined whether the costs of pollution abatement are to be borne by the polluter or by the downstream water user. If the cost is to be borne, even in part, by the water utility, its customers must then accept rate increases to support the necessary research for new treatment methods, new plant construction, and increased operating costs. If the public is made aware of these facts and of the seriousness of the problem, the steps necessary to solve it may yet be taken before it is too late.

Significance of Pesticides in Water Supplies

—Richard L. Woodward—

A paper presented on Apr. 25, 1960, at the Canadian Section Meeting, Buffalo, N.Y., by Richard L. Woodward, Chief of Eng., Water Supply & Pollution Control Research, Robert A. Taft San. Eng. Center, USPHS, Cincinnati, Ohio.

SINCE World War II, the pesticide chemical industry has undergone a revolution. Before the war, the principal insecticides were either inorganic compounds, such as the arsenic compounds, or naturally occurring organic compounds, such as pyrethrum, rotenone, and nicotine. Herbicides were primarily copper sulfate, arsenic compounds, and other inorganic materials. The introduction of DDT and 2,4-D marked the start of a tremendous surge in the development and use of new products that have had far-reaching effects on public health and agriculture. An almost bewildering array of pesticide chemicals are on the market today, and new ones are being developed and introduced continuously. Several hundred pesticide chemicals are now in use, and the number of products and formulations registered under the federal Insecticide, Fungicide, and Rodenticide Act exceeds 90,000.¹

Water utilities have not felt much impact from these developments to date, but concern is expressed in many quarters about the water quality problems that these new chemicals may cause. It is virtually certain that any chemical in widespread use will get into streams in some concentration. Some of these chemicals are quite resistant to biologic attack and quite

toxic to humans. Comparatively little is known about ways to remove these chemicals from water. There is a need to anticipate the problems that may develop and to find solutions for these problems.

Water utilities are, themselves, substantial users of pesticide chemicals. As operators of water reservoirs, they often need effective control of algae, insects, and vegetation in and around reservoirs. For dealing with these problems, some of the newer pesticide materials may be superior to the old ones.

Pesticide Production and Use

Statistics on production and use of pesticide materials leave something to be desired, but one can get some appreciation of the amounts of the synthetic organic pesticides produced and sold in the United States from reports of the US Tariff Commission² and from annual reviews by Shepard.³ In Canada, the Dominion Bureau of Statistics⁴ is a source of similar information.

Table 1 shows 1958 sales of synthetic organic pesticide chemicals in the United States.² The data show the still dominant position of DDT among the insecticides and 2,4-D among the herbicides, although they were the earliest to be introduced. Although about 25,000,000 lb of arsenic compounds

and an equal amount of copper sulfate are used annually, the organic pesticides have displaced these chemicals for many uses.

The great increase in production and use of the newer pesticide chemicals came in the late 1940's and early 1950's. Between 1955 and 1958, production in the United States showed only moderate increases. Some of the newer chlorinated-hydrocarbon insecticides have shown marked increases in sales in recent years.

Canadian sales of pesticides amounted to about \$25,000,000 in 1959, as compared with \$19,000,000 in 1954. On a per capita basis, this is comparable with United States sales. As in the United States, use in Canada of the older inorganic and botanical products is decreasing, and use of the synthetic organic compounds is increasing. In dollar value, dieldrin is the largest selling insecticide, followed by DDT and aldrin. Similarly, of the herbicides and fungicides, 2,4-D and the carbamates account for the largest volume of sales.

A comparison of the data in Table 1 with those for a more familiar chemical may make them more meaningful. Water supply personnel have heard much in recent years about synthetic detergents and of the dominant surface-active agent, alkyl benzene sulfonate (ABS), in these products. In 1958, United States sales of ABS were about the same as the total sales of all synthetic organic pesticides.

An interesting computation can be made to relate these quantities of chemical to water. The annual runoff of all streams in the United States averages about 1,160 bgd. If the 466,000,000 lb of pesticides sold in 1958 were diluted by the average annual runoff of all streams, the resulting pesticide concentration would be about

0.13 ppm. This computation is only meant to convey an idea of the order of magnitude of the quantities involved. Only a portion of these chemicals reaches watercourses. On the other hand, the usage of pesticides is far from uniform, either geographically or in time.

Concentration of Pesticides in Water

Comparatively little is known about the concentrations of pesticide chemicals now in water. Middleton and Lichtenberg⁸ have found DDT in a number of large rivers in the United States,

TABLE 1
Sales of Synthetic Organic Pesticide Chemicals in the United States, 1958

Pesticide	Sales 1,000,000 lb
DDT	145
Other chlorinated-hydrocarbon insecticides	133
Parathion and methyl parathion	9
Other insecticides	13
Fungicides and soil fumigants	114
2,4-D and derivatives	26
2,4,5-T and derivatives	4
Other herbicides, rodenticides, and soil conditioners	22
<i>Total</i>	<i>466</i>

including the Detroit River; the Missouri at Kansas City, Kan.; the Mississippi at Quincy, Ill., and at New Orleans; and the Columbia at Bonneville, Ore. The concentrations at these places were approximately 1-20 ppb. Aldrin, another chlorinated-hydrocarbon insecticide, has been found in the Snake River at Pullman, Wash., in a concentration of approximately 1 ppb. These concentrations caused no perceptible damage.

Some of the chlorinated-hydrocarbon insecticides are extremely toxic to fish and other aquatic life. A number of fish kills have resulted from their use

in controlling agricultural and forest insects. DDT, toxaphene, dieldrin, and endrin have caused such incidents in the United States and Canada.^{6,7} It has seldom been possible to measure the concentration of the insecticide in the water at the time of these incidents, but from a knowledge of the toxicity of the chemicals, estimates can sometimes be made. In a field test, runoff from an area treated with 4.66 lb/acre of dieldrin was found by fish bioassay to contain about 0.13 ppm of the insecticide.⁸ A chemical determination showed 0.10 ppm.⁹

Fate of Pesticides

It is difficult to make categorical statements about such a varied group of chemicals as the pesticides, but some characteristics can be pointed out. The bulk of pesticides is used in controlling agricultural pests and is applied to vegetation or to the soil. Some pesticides are incorporated into plants or remain on them as residues. A part is lost by volatilization; a part is fixed in the soil. Biologic attack or oxidation by natural agents alters many of the products. A part is leached from the soil by rainfall or irrigation water and either appears as runoff or percolates into underground water.

The extent to which each of these transport mechanisms takes place varies with the particular chemical. The soil fumigants are, of course, largely volatilized. The chlorinated hydrocarbons as a class are not degraded very quickly by biologic life, but some of them are very tightly adsorbed on soil particles, while others are much more readily leached from the soil.

A great deal of research has been done on the fate of various pesticides. This research has been oriented pri-

marily toward the problems of the user of the products. Very little research has been directed toward associated problems of water quality. Much of the agriculturally oriented research does provide useful qualitative information bearing on present problems, but many questions remain to be answered, and research on these questions is needed.

Toxicity of Pesticides

The only reason for using any of the pesticide chemicals is that they are toxic to something. Although many of them are not particularly toxic to man, others are among the most toxic materials in common use. This has led to strict regulation of pesticides and to widespread and intensive toxicologic studies of the materials.

In addition to the requirement that pesticide formulations be registered and properly labeled, the US Food & Drug Administration establishes tolerance limits for pesticide residues in or on food products.¹⁰ These tolerances are established at levels no higher than necessary in the light of good agricultural practice in the use of the pesticide, and at levels considered safe for ingestion. This practice results in tolerance values that may be different for the same chemical on different crops. For instance, although 7 ppm DDT is permitted on many foods, a zero tolerance has been established for DDT in milk. For some chemicals, such as endrin and aminotriazole, zero tolerances have been established for foods generally. A few materials, such as sulfur, have been exempted from tolerances.

No official limits have yet been placed on concentrations of pesticides in water in the United States, except for limits on arsenic, selenium, fluoride, and lead. This is not surprising, in

view of the fact that the current USPHS Drinking Water Standards date from 1946. The standards are now being revised, and consideration is being given to the inclusion of limits on some of the chlorinated-hydrocarbon insecticides and some of the organophosphate insecticides. There is a general feeling that one should not permit as high levels in drinking water as are permitted on certain food products. But the establishment of a zero tolerance for a particular chemical in water is not feasible. For residues on food, a zero tolerance generally requires that the product not be used on the crop during the period when it might result in contamination of the harvested portion of the crop. Such a limit in water would be tantamount to a prohibition of the use of that material.

In general, these insecticides are undesirable additives to water. Every effort should be made to keep their concentration not only below the threshold of any toxic effect but also as low as is reasonably possible.

Taste and Odor

In addition to their toxicity, a number of the synthetic organic pesticides cause highly objectionable tastes and odors in water. Several of such chlorinated hydrocarbons as toxaphene and endrin have threshold odor concentrations on the order of a few parts per billion. Information is not available on the taste and odor threshold concentrations of most of the pesticides or of their reaction products when chlorinated. Solvents used in many of the pesticide formulations are also highly odorous.

Pesticides in Water Supply Practice

Many water utilities have problems of insect and weed control related to

their operations. It is obvious that the greatest caution should be used in dealing with these problems, lest more difficult problems be created through the use of an unsuitable control method. Clearance with the appropriate health authorities is necessary before any new pesticide formulation is used, and consultation with experts on the problem will generally be worth the cost.

As a rule, materials that are not persistent are to be preferred to those that are persistent. Another important consideration is the ease with which the materials can be removed by water treatment processes. Unfortunately, knowledge of these matters is rather fragmentary. It should not be forgotten that public water supplies are used for watering lawns, shrubs, and flowers, and for filling aquariums. It will certainly not help a water utility's public relations if residues of herbicides or insecticides kill the customer's gladioli or guppies.

Another way in which water utilities may be affected by pesticides is through their use in fishery management. It is sometimes desirable to kill all fish in a lake, reservoir, or stream system to get rid of coarse fish prior to introducing more desirable species. Rotenone has been the traditional fish poison used, but toxaphene is considerably cheaper, quite effective, and has been used extensively.

Where water supplies are involved, the use of toxaphene should be discouraged. Toxaphene is quite persistent and highly odorous. It can be removed from water by using activated carbon, but other standard treatment procedures are ineffective. Activated-carbon treatment may be necessary for a long time because of the persistence of toxaphene. Rotenone, on the other hand, is fairly rapidly oxidized natu-

rally, and special treatment is not required for very long. The solvents employed in some of the rotenone preparations used as fish poisons may be more troublesome than the rotenone.

Research

The need for research on a number of aspects of the problem of pesticides in water supplies has been mentioned previously. Some of this work is going on. USPHS, through its regional office in Atlanta, is cooperating with several of the southeastern states in a field investigation of the insecticide pollution problem in that region.

Laboratory and pilot plant studies of the persistence of selected pesticides in water and the removal of these chemicals by water treatment are being carried on at the Robert A. Taft Sanitary Engineering Center of USPHS at Cincinnati. Among the materials being investigated are lindane, parathion, 2,4-D, endrin, and several fish poison preparations containing rotenone or toxaphene. Work is under way or proposed on several other chemicals, including aminotriazole and one of the carbamate fungicides. A continuing program for investigation of the effects of pesticides on fish and other aquatic life has been carried on at the Taft Center for a number of years.

Little or nothing is being done on several important aspects of the problems regarding pesticides and water quality. One area that has been almost totally neglected is that of ground water pollution. Little attention was paid to organic chemicals as ground water pollutants until recently, when an increasing number of incidents of ground water pollution by synthetic detergents has directed attention to this problem.¹¹ The concentration of detergents found in many wells has been

much higher than was normally found in surface waters. There is reason to suspect that similar situations are developing in localized areas where pesticide use has been heavy and continued, where the soil is comparatively porous, and where the ground water table is shallow. Investigations to determine the concentration of pesticides in such situations are needed particularly.

Utility Action

A water utility should be informed of pesticide use on the watershed or recharge area from which its supply is drawn. If this is a very large area, as with a supply from a large river or one of the Great Lakes, this may not be possible. But at the same time, the problem is not likely to be as important as it is where a considerable part of the watershed may be treated with pesticides. To the degree that the water utility can influence pesticide use, it should do so, just as it should take other measures to protect the quality of its supply. It should also equip itself with facilities and supplies to deal with foreseeable problems.

Unfortunately, precise and simple analytic procedures for the various pesticide chemicals in water are not available to the water plant operator for monitoring the quality of his supply or the performance of his treatment plant. The fact that some of the insecticides are so poisonous to fish can be used to detect the presence of these compounds in water and to estimate their concentration. Pickering and Henderson¹² have outlined a relatively simple procedure for doing this. This procedure is limited in its application to a comparatively few of the pesticides. Rosen and Middleton⁹ have described procedures for eight of the chlorinated insecticides that are sensitive in concentrations measured in

parts per billion. The procedures are lengthy and require infrared spectrophotometry in the final identification.

Summary

At present, the significance of the potential threat of pesticide chemicals in water supplies cannot be defined clearly. Additional investigations and research are needed to do this and to develop methods of dealing with the problems that appear likely to arise.

These chemicals present a type of water pollution problem that cannot be controlled in the same way as sewage and industrial waste discharges. Widespread use of insecticides is a fact, and the economic need for them is so great that larger quantities of insecticides can be expected to reach water supplies. If they are present in significant concentrations, they must be removed by treatment.

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Automatic Rubber Diversion Dam in the Los Angeles River

N. M. Imbertson

A paper presented on May 17, 1960, at the Annual Conference, Bal Harbour, Fla., by N. M. Imbertson, Engr. in Charge, Water Plant Operating Div., Dept. of Water & Power, Los Angeles, Calif.

THE Los Angeles Department of Water and Power has for 45 years maintained a 30-acre spreading ground located adjacent to the south side of the Los Angeles River between the Walt Disney Studios and Forest Lawn Cemetery.

Over the years the department has been diverting water from the river by means of a 5-ft wooden dam, which raised the water sufficiently high to take it through gates and carry it over the spreading grounds. Any surplus clear water coming down the river was diverted to these spreading grounds and served to recharge the underground water, to be removed later by pumping.

Diversion Problems

Whenever there was more than 40-50 ppm silt in the water, the spreading grounds would silt off and would require cleaning after 6-7 weeks of operation. If there was 400-500 ppm turbidity, the basins would choke off even sooner. The normal method of operation during such periods was to bypass the muddy waters, because maintenance costs averaged around \$6,500 per cleaning of the 30-acre basin. Also, because the Los Angeles River is a flood control channel, the

wooden dam diversion structure was designed to wash out under any major runoff down the Los Angeles River. The net result was that the percentage of water reclaimed in the spreading basin was rather small as compared to what could be saved if the dam could be operated whenever there was water available in the river.

Treatment of Recharge Water

In 1956, at the American Mining Congress, a large chemical manufacturer was advertising an organic polymer* they had been using in the mining industry for ore processing. This remarkable product would settle out colloidal materials in water with as much as 1,000-2,000 ppm turbidity in a matter of minutes. In fact one of the flocculant's startling properties was that the more colloidal material or fine silts in suspension the faster it seemed to settle material out of the water.

It was obvious that considerable additional quantities of water could be put into the spreading grounds if there were a way to remove the silt before the waters were spread on the settling basins. The chemical manufacturer

* Separan 2610; a product of Dow Chemical Co., Midland, Mich.

thought it could desilt the water with its product. Accordingly, the department worked with the manufacturer in setting up a pilot plant, and results indicated that the product would obtain the required results at a reasonable cost.

capacity of 40 cfs. The plant was located at this point because:

1. There was an access road down to the river bottom.
2. There exists at this point what is known as a pilot channel in the bottom of the Los Angeles River. This

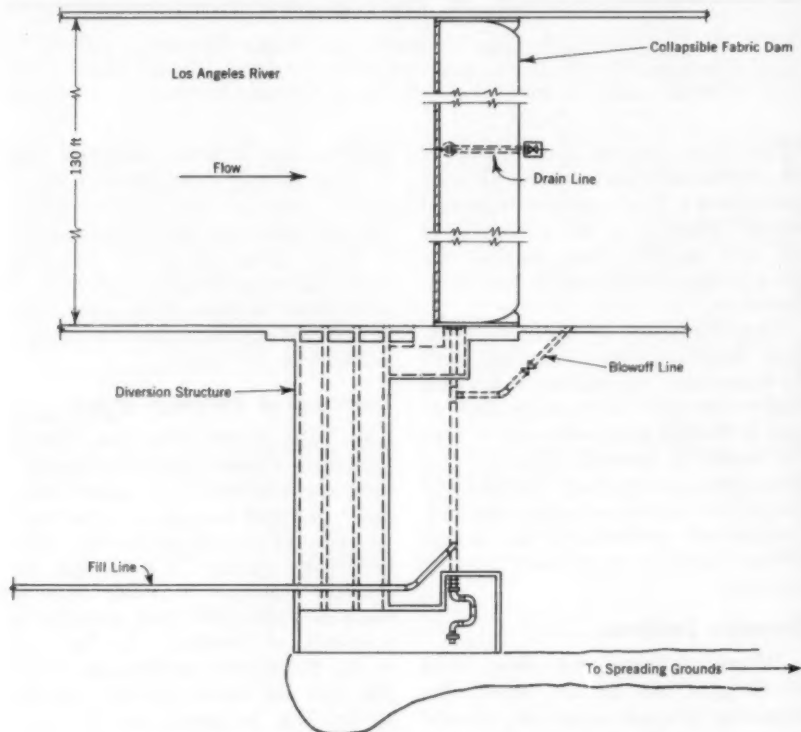


Fig. 1. Plan View of Automatic Rubber Division Dam

The fabric for the dam is $\frac{1}{8}$ -in. thick nylon-reinforced neoprene.

At the manufacturer's recommendation, the department picked a spot in the Los Angeles River $2\frac{1}{2}$ mi above the diversion gates leading to the spreading grounds and erected a treatment plant on the adjacent bank with a ca-

pilot channel is 10 ft wide and carries a maximum of 80 cfs of water. This provided an ideal point at which to add the flocculant solution.

3. Being added $2\frac{1}{2}$ mi above the wooden dam which formed a settling

basin, there was ample time for the flocculant to mix with the water.

After the plant was set up, a test run was made, starting Dec. 11, 1956, and continuing for 22 days. During the period of the test, an average of 12.8 cfs of water was treated and placed in the spreading grounds (563 acre-ft, total). At the end of the 22-day period, the basin was drained and inspected. The results of the tests indicated that:

1. The average spread water carried only 15 ppm of turbidity, ranging from 7 ppm to 20 ppm.
2. Practically no silt accumulated in the spreading basins.
3. The desilting operation was being accomplished at a total cost of \$1.50 per acre-ft of water treated.
4. Because the Flood Control District estimates that the water spread is worth \$15 per acre-ft, the net gain for the test was \$7,300; and had this water been put in without treatment, there would have been a maintenance cleaning cost of about \$6,500, as well as a loss of water during cleaning (approximately 3 weeks).

Cleaning of Basins

Because the basins would have to be cleaned only because of algae problems, serious thought was given to the development of a special method of cleaning. The result was a specially built scraper which makes possible the removal of the skin of algae together with not more than 1 in. of surface material. On the first try, this new machine cleaned the 30-acre spreading grounds for \$1,300, instead of \$6,500, so that the machine, which cost \$5,000, paid for itself on the first cleaning job. The machine consists of a 5-yd carryall with paddle wheels

just above the blade which pushes the algae or silt back into the body of the carryall. Ahead of this is a roller which maintains the carryall on an even keel. In order to make this machine practical, it was necessary to grade the bottom of the basins and remove the large boulders.

Need for New Type of Dam

After 6 months of operation with use of the flocculant, it became plain that there were no serious problems in treatment. It also became clear that if there were a type of diversion dam which could be kept in operation during the stormy season, it would be practical to divert even the muddy storm waters into the channel leading to the spreading basins 2,700 ft downstream. By erecting another treatment plant between the diversion gates and the channel leading to the spreading basin, these waters could be treated in the channel, the silt could be settled in a separate settling basin, and the water could then be led into the spreading basins for ground water replenishment.

Accordingly, it was decided to spend some money to design a diversion works which would not wash out in the winter, and plans and estimates were made to place a Tainter gate across the river; however, this met with objections from the Army Corps of Engineers, because it would involve a center pier that would tend to obstruct the full flow of the river, which at this point was in the neighborhood of 45,000 cfs. The estimated cost of the Tainter gate was \$120,000.

Rubber Diversion Dam

One solution was to construct a diversion dam of some material which

would withstand the abrasion of waterborne rocks and sand, would collapse so flat on the concrete channel floor that roots, floating trees, and general trash would not become entangled, and yet could be easily raised into a dam after the flood.

Rubber and synthetic rubber are such materials. They are flexible, can be made to collapse, withstand abra-

Prototype Dam

A prototype automatic dam was constructed at one end of the wooden dam in the latter part of 1957. The prototype was 20 ft long, operated at a height of 5 ft, and was fastened to the reinforced-concrete river channel floor at the upstream edge with steel anchor bolts. One end was fastened to a bulkhead at the end of the wooden



Fig. 2. Rubber Diversion Dam in Operation

In this view the fabric dam is shown fully inflated. When deflated, the fabric lies flat on the channel bottom.

sion very well (an automobile tire is an example), can be reinforced with fabrics to give them strength against tearing, and are not excessively expensive. Rubber tends to weather and check from sunlight and smog, however. Therefore, neoprene, a synthetic rubber which weathers well, was selected. It was reinforced with nylon fabric for maximum strength and for weathering qualities.

dam and the other end was fastened to the concrete wall of the channel.

Various methods of attaching the ends were tried, the principle of automatic collapse was perfected, and, because of the many severe storms in the winter of 1957-58, the many aspects of the strength of the materials, abrasion resistance, methods of installation, and operation of the dam were tested.

Full-Size Dam

The full-size dam was then designed as a large tube or bag, tear-drop shaped in cross section, stretched across the river bottom and anchored to the bottom at the upstream edge with stainless-steel bolts set into the concrete floor of the channel (Fig. 1). The ends of the tube were extended up the vertical walls of the channel and also anchored with bolts. The tube is 30 ft in circumference and 150 ft long. The river channel is 130 ft wide, so the tube extends up each wall 10 ft (Fig. 2).

The nylon-reinforced neoprene fabric* is $\frac{1}{8}$ in. thick, and a 1-in. wide strip of the material will support more than 500 lb before breaking. The tube is inflated with water from a pipe connection through the channel wall.

Operation of Dam

To make the dam collapse automatically in case a storm causes flooding conditions in the river channel, a siphon was installed in the spreading basin channel and connected through an 18-in. water pipe to the water inside the dam. Thus, the pressure of the water in the dam is transmitted to the water in the pipe, which terminates in a simple siphon of a regulated height such that if the pressure within the dam attains a predetermined figure, the siphon will prime and deflate the dam. The dam can be inflated to any desired height up to approximately 6 ft, depending on the adjusted height of the siphon, which is designed to rotate 90 deg.

* The fabric used in the dams was manufactured by Firestone Tire and Rubber Co., Akron, Ohio.

When a storm causes the water level in the river to rise, the excess water flows over the top of the dam. The weight of this water on the top of the dam increases the pressure on the water inside the dam. When this pressure gets great enough, the water in the dam charges the siphon and the dam starts to deflate. This continues until the dam is completely collapsed.

When completely collapsed in a storm, the fabric lies flat on the bottom of the river channel and up the side walls, and all debris is carried over it by the water. When the storm waters subside, the dam is reinflated with water from a tank located on the bank of the river and then goes back in operation.

The treatment plant constructed between the diversion dam and the spreading basins permits treatment of storm waters during both winter and summer, except for a period of perhaps a day or so when, because of storm conditions, the rubber dam is deflated.

The prototype dam has been reconstructed as a diversion dam in the spreading grounds. It permits diversion of water to either the upper or lower basin, so that during low flows one group of basins can be cleaned while the other is in operation.

Other Applications

Since the completion of the large dam in 1959, there have been hundreds of inquiries. Foremost among them are requests relative to the feasibility of:

1. Temporarily raising the spillway lip of dams, permitting the storage of 2-4 ft of extra water in the reservoir. This is made possible because the collapsible feature of the dam guarantees

the original spillway capacity of the main dam if it is required.

2. Using the dam in irrigation works for diversion, replacing flashboards and sluice gates.

3. Using the dam to gain 1-10 ft additional head, replacing stoplogs on ogee or overflow type dams for hydroelectric plants, projects across rivers, or other applications.

4. Use of rubber dams for diverting runoff flows from streams for irrigation of adjacent ranches.

5. Use of rubber dams for levee construction. A collapsible dam to put flood waters where they would do the

least damage could well save the city or cities below.

6. Use of rubber dams to prevent ocean tides from running up rivers and, at the same time, to intercept and perhaps put to good use the fresh waters coming down the rivers.

7. Use of the dams to create lakes or fish ponds for resorts and other purposes.

8. Use of the dams to replace Bascule and Tainter gate installations in certain instances.

9. Use of the dams in skimming processes or pulp handling.



Paints and Resins for Steel Tanks

—James O. Jackson—

A paper presented on May 18, 1960, at the Annual Conference, Bal Harbour, Fla., by James O. Jackson, Chairman, Joint Com. 8710 J—Steel Standpipes and Elevated Tanks, and Pres., Engineering Development Co., Coraopolis, Pa.

JOINT Committee 8710 J—Steel Standpipes and Elevated Tanks, is preparing a complete revision of AWWA Standard D102, which deals with the painting of elevated tanks and standpipes. Many improved paints that depend on the use of new synthetic resins have become available since the publication of the previous standard.¹ Therefore, the properties of resins and their influence on the performance of paints merit discussion here.

Red-Lead-Flaxseed Oil Paints

The term *paint* is a very general one and includes almost any fluid that may be applied to a surface and that dries to form a protective film. A mixture of finely ground red lead and flaxseed oil has been used in the past as a paint for both the interiors and exteriors of steel tanks. Flaxseed oil is extracted from flaxseed and has valuable properties. It penetrates into porous surfaces, such as those caused by rust and mill scale, and it dries to form a hard film. Flaxseed oil dries by the absorption of oxygen from the atmosphere, causing a chemical change or polymerization of the oil into a solid substance. This drying requires an appreciable length of time, from several days to several weeks, depending

on the temperature, the amount of ventilation, and the thickness of the paint coat. The drying time can be accelerated by the addition of chemical driers or by preprocessing the flaxseed oil by heating and air blowing.

Although flaxseed oil alone may be used as a protective coating, its service life is short. Finely ground red-lead pigment extends the life of the oil very considerably by rendering the film opaque to destructive ultraviolet light and by providing a rust-inhibiting chemical at the surface of the steel. A large amount of red lead may be mixed with a small amount of flaxseed oil to provide a thick paint film after the oil has dried. It has been found that the period of protection afforded by a paint depends largely on the thickness of the paint film.

Although a mixture of red lead and flaxseed oil makes a good paint, it has many inherent defects. If used on the outside of a tank, the red color soon fades, causing the paint to have a bad appearance, although it may continue to protect for some time. For this reason, it is customary to use the red lead on the outside only as a primer and to apply over it one or more coats of paint containing iron oxide or black or aluminum pigments that do not fade. Red-lead-flaxseed oil

paints used on the inside of tanks where they are immersed in water usually fail within a short time from excessive blistering. This tendency to blister, although not completely understood, is known to depend on the porosity of the paint film and the chemical nature of the ingredients in the film. The permeability of a dried linseed oil film may be greatly reduced by the addition of resins. For example, the permeability of a dried linseed oil film that has a relative permeability to water vapor of 0.895 may be reduced to 0.302 by the addition to the linseed oil of a 50 per cent kauri gum resin. Unfortunately, most suitable resins are brittle, so that the addition of any appreciable amount of resin to the oil results in a brittle paint film. This brittleness may be compensated for by the addition of plasticizers that, by incorporating a thick viscous material into the film, act to reduce the inherent brittleness of the resin.

The very desirable wetting characteristics of linseed oil enable it to tolerate some rust and mill scale on the steel surface and act to bind the paint film to the steel surface through the rust and mill scale. The result is that, during aging, the paint film is not lifted from such areas by the continuing oxidation of the steel and the formation of resulting gas pressures.

During the past 50 years, the materials and techniques just described resulted in the development of paints for the exteriors and interiors of steel elevated tanks and standpipes. These paints—mostly based on linseed oil vehicles, on red-lead pigments in the primers, on iron oxide, carbon black, and aluminum in the exterior finish coats, and on suitable combinations of resins in the interior or water-

immersed surfaces—have been the standard of the immediate past. The service life of these paints has not been good when measured against present standards. These types of paints for exterior surfaces have given protection for 1–3 years when repainting was necessary because of the bad appearance of the structure caused by paint scaling off and leaving rust blisters and other unprotected areas.

On the interiors of the tanks, the service life has been even shorter. With these types of paints, blistering usually begins within a few months. Frequently, the interior surfaces are so badly blistered that repainting becomes necessary within from less than 1 year to 1.5 years after the tank is put in service. Repainting the tanks with the materials described is a costly operation when the tank surface is partially pitted with large numbers of rust blisters and has areas where the paint adheres tightly over mill scale. Cleaning by hand methods is inadequate. Cleaning with power sanders and other similar tools is possible, but this requires the removal of so much of the surface that the cost of the process is excessive. Cleaning by sandblasting has been found to be the best method.

Phenolic Resins

During the past two decades, there has been a broad development in the creation and use of plastics. The new plastic materials are nearly all based on the chemical creation of new resins, which have such a variety of physical and chemical properties and which may be modified in such infinite variety that many types of paints may be made employing them. Some of these resins have very desirable properties that can be utilized in paints to protect steel tanks.

The first synthetic resin of commercial importance was a synthetic phenol formaldehyde material.* The phenolics now include a large family of reaction products of phenols and aldehydes comprising complex polymerized products. These are generally "thermosetting"—that is, they set and harden on application of heat. They are relatively inflexible and inert, except in the presence of strong alkalis. By substituting other phenols in place of the carbolic acid generally used, "thermoplastic" oil-soluble resins may be obtained. Thermoplastic resins are those that become fluid or plastic when they are heated; in other words, they melt. The oil-soluble thermoplastic resins are generally soluble in aliphatic hydrocarbons and are used extensively in making varnishes that are excellent as spar varnishes or for use in paints to protect metals. These varnishes are very durable in water, have good flexibility, high resistance to severe environment, and are superior to the oils with which they are combined. By varying the conditions under which the polymerization action occurs in making a phenolic resin, a thermosetting resin, which is also oil soluble, may be obtained. The phenolic resins, because of their impermeability, have been used with excellent results on the interiors of water tanks.

Alkyd Resins

Another group of synthetic resins known as alkyd resins, commonly referred to as alkyds, are made from polyhydric alcohol and a polybasic acid. These may be modified with

resins, fatty oils, or fatty acids. Alkyds are generally made from glycerin and phthalic anhydride and usually modified with drying oils or fatty acids. A combination of glycerin and phthalic anhydride results in a product known as glyceryl phthalate. In making alkyd resins, glycerin or some similar polyhydric alcohol, such as pentaerythritol, is reacted with phthalic anhydride, fatty acids, and drying oils to produce an alkyd of the desired properties.

The use of alkyd resins in the paint industry is rapidly increasing, because the alkyds make paint films durable and quick drying. Their use results in a film much harder than the ordinary oil paint film. Their quick-drying properties are of great value where mass production operations are necessary or when little time is available for drying. They provide good gloss retention, but eventually fail by checking rather than by gradual weathering because of their brittleness.

Because of the poor wetting characteristics of alkyd paints, they are unsuitable for use over surfaces that contain appreciable amounts of rust or mill scale. It is therefore necessary to remove rust and mill scale by pickling or blast cleaning before alkyd paints are applied. Alkyd paints are not as suitable for interior or water-immersed surfaces as are other paints, but they are excellent for exterior weather-exposed surfaces. They are available in a variety of colors, including tints and aluminum.

Vinyl Resins

Another family of synthetic resins that has made possible extremely durable paints, especially for tank interiors, is the vinyls. The vinyl resins used

*Bakelite, made by Bakelite Co., a Division of Union Carbide Corp., New York, N.Y.

in protective paints for metals are made from vinyl compounds that are copolymerized. The main constituents are vinyl chloride and vinyl acetate. This copolymer is a relatively inert, insoluble resin of water-white color which has outstanding durability. It is soluble only in special types or combinations of solvents. Such vinyl resins have poor adhesion to metal. To improve the adhesive quality, other constituents, such as maleic acid, are added. The resulting resins have good adhesion to steel, to other vinyls, or to other paints. These resins are different from the ordinary paint vehicles because they do not oxidize. They dry solely by solvent evaporation, in the same manner as lacquers. Because they are usually dissolved in solvents with low boiling points, the vinyl resins are extremely fast drying. Their poor wetting ability, however, limits their use to clean steel. They are used in solutions with low concentrations of solids.

The vinyl resins provide an interior tank paint that, when applied over pickled or blast-cleaned surfaces, will result in many years of service with practically no maintenance expense. Vinyl paints have been used on the interiors of steel tanks for at least 10 years with very little deterioration. The hundreds of tanks painted with vinyls have proved the effectiveness of the material.

Chlorinated Rubber

A synthetic resin made by chlorinating natural rubber has resulted in greatly improved paints. Chlorination, when carried out to the extent necessary to make the resin, completely alters and changes the rubber-like properties. The resin is a white,

fluffy powder that is hard and brittle. It has high chemical resistance, particularly toward acids and alkalis. When properly formulated, it makes a good priming paint or a good top coat paint. Chlorinated-rubber paints have poor wetting ability on dirty or rusted steel. Because of the brittleness of the chlorinated resin, it requires plasticizers when made into a paint. Aromatic solvents are necessary for dissolving the resin.

Exposure tests of chlorinated-rubber paints have shown that these paints may be excellent for both the interior and exterior surfaces of steel tanks. Detailed formulations are being developed from research testing. When the best formulations are developed, actual full-scale tests will be made to evaluate the worth of chlorinated-rubber paints for both the interiors and exteriors of steel water tanks.

Epoxy Resins

A family of synthetic resins that is showing excellent promise is the epoxy resins. These are a new development in synthetic resins that are somewhat similar to the phenolic resins, in that they are a condensation product of a polymerization process in which water is eliminated. Epoxy resins are made by combining polyphenols and epichlorhydrin. In their unmodified form, they are thermosetting—that is, they require heat or baking to make them hard. In that form, they are generally unsuitable for structural steel finishes. A cold curing formulation, however, can be made by adding a curing agent to make the resins set hard at normal temperatures. Various organic amines are used to effect this hardening and are added prior to application. The resulting product may

be applied in very thick films, because it polymerizes without oxidation and without the need of oxygen from the air. Epoxy resins may also be reacted with fatty acids. The resulting ester has many of the properties of epoxy resins. By the addition of the oil constituent, the resins lose much of their high chemical resistance. The result, however, is a product that will oxidize and air dry to form a solid film.

Paints based on epoxy resins for use on both the insides and outsides of steel tanks have been under experimental investigation for at least the past 5 years. To the author's knowledge, no specifications for materials proved in research testing have been made available to the public. Proprietary or secret formulations have been made; these have been applied to steel tanks for at least the past year and, for this short time, are apparently giving good service. It is expected that formulations will be available for epoxy resin paints, particularly for the interiors of water tanks, within the next few years. These formulations will have been proved by actual use to provide satisfactory and, perhaps, even outstanding performance.

Silicones

Silicones are a new development in resins for paint. They are somewhat similar to hydrocarbon compounds, but silicon and oxygen replace some carbon atoms. The price of silicone resins is very high, and conditions for their use must be very severe or very special to justify the present cost. In high-temperature protective paints for metals, the use of silicones is increasing. In fact, silicones are one of the few materials that show promise when exposed to intermediate or high tem-

peratures. Silicone resins can be used alone or in combination with other resins, such as alkyds, to make moderately high-temperature paints of good durability. This type of coating retains its stability at temperatures as high as approximately 500°F. Ordinary paints tend to decompose at lower temperatures. Decomposition of even the silicone resins begins at a temperature higher than 550°–600°F. The durability and corrosion resistance of the final product after it is heated to temperatures near 1,000°F remain to be definitely established.

Nitrocellulose

A number of cellulose derivatives are used in protective coatings. The best known of these is nitrocellulose, made by treating cellulose with nitric acid. Nitrocellulose is usually plasticized and pigmented to make a lacquer. Powerful solvents are necessary because of the relative insolubility of the nitrocellulose. Other available cellulose compounds are ethyl cellulose, cellulose acetate, and benzyl cellulose. These products are not of great use to the structural-steel industry, although there is a growing trend toward using lacquers on steel, particularly since the development of the hot-spraying lacquer process that has eliminated much of the wasted solvent.

Styrene

Styrene is a reactive product that may readily be polymerized to make a synthetic resin or copolymerized with other materials. Polystyrene, made by polymerizing styrene, is not of great interest in the metallic-coating field. Styrene may be reacted with butadiene to make a synthetic rubber. A similar reaction results in styrene-butadiene

copolymers, made by varying the ratio of styrene to butadiene. This resin has properties somewhat similar to synthetic rubber, but styrene-butadiene copolymer paints are vastly different from synthetic rubber. The resin is particularly useful because of its resistance to alkalis and acids.

Coumarone-Indene Resins

Coumarone-indene resins are produced from coal-tar byproducts. They vary considerably in their properties and may range from solid to liquid, because their melting points vary. They have fairly good alkali and acid resistance and are used extensively in making varnishes, particularly for aluminum mixing. These resins are also used for primers to be immersed in sea water.

Inorganic Coatings and Resin Mixtures

A number of inorganic coatings very similar to paint are available. These are based on mixtures of zinc dust and sodium silicate solutions. In the process of baking or curing, the silicate is believed to react with the zinc to form the protective coating. These products show promise of being very useful on steel exposed to severe surroundings, but they are quite brittle. They must either be baked or cured at low temperatures by application of a suitable catalyst.

Almost an infinite variety of paints may be produced by mixtures of different resins. For example, a coal-tar epoxy mixture is believed to produce a paint for severe corrosive conditions.

Other Synthetic Resins

There are many other synthetic resins available, but these have rather

limited use in structural-steel paints. Many of these resins are used in large quantities in baking-type enamels or other types of finishes for specialized application. Urea or melamine resins are formed when urea or melamine is reacted with formaldehyde to make thermosetting resins. Acrylate and methacrylate resins are used to make lacquers because of their water whiteness, transparency, and general inertness. They have good resistance to acids and alkalis but are not yet used in pigmented structural paints. A number of coatings are available which are based on neoprene. These are available in two types that are applied in the same manner as conventional paints. The first type must be mixed with an accelerator and a vulcanizer before application, in order to make the material cure properly. The second type of neoprene coating is similar to paint and requires no added accelerator or vulcanizer prior to use. Petroleum resins are derived from petroleum residues, but they are not petroleum asphalts. They may be either of the oxidizing or nonoxidizing type. They are incorporated into varnishes and are being used particularly in aluminum mixing varnish. Many other synthetic resins are available. Some of these, such as cycloized rubber, are based on natural rubber. Others are polymers or copolymers of organic compounds—polyethylene, polytetrafluorethylene, and nylon, for example. Many of these are available in the form of coatings or linings rather than as paints. Bituminous resins cover a wide variety of products ranging from solids to liquids. Their classification and definition and the pertinent terminology are vague and confusing. These products may be natural or synthetic. Bituminous

resins or bitumens are used extensively in the form of cutbacks, either filled or not filled, or associated with drying oils to make cold-applied or hot-applied coatings. These may air dry, or, in some instances, particularly in Japan, they may require baking or stoving. Bituminous materials include bitumens, asphalts, tars, and pitches. These materials for the hot- and cold-applied

AWWA tar coatings are used for both pipe and tank interiors.

Reference

1. Tentative AWWA Standard for Painting and Repainting Steel Tanks, Standpipes, Reservoirs, and Elevated Tanks, for Water Storage—AWWA D102. Am. Wtr. Wks. Assn., New York (1955).

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Controlled Deposition of Protective Calcite Coatings in Water Mains

Robert F. McCauley

A contribution to the Journal by Robert F. McCauley, Assoc. Prof. of Civ. and San. Eng., Michigan State Univ., East Lansing, Mich.

IN two articles published recently,^{1,2} the author described a process that he developed for the controlled deposition of protective calcite deposits in water distribution lines. In these articles, the results of a series of experiments with this coating process were related to the hydraulic and chemical principles that govern the deposition rate and crystalline structure of the calcite coatings.

Until recently, experimental work on this process has been with $\frac{3}{4}$ -in. black-iron nipples and with thin, flat, 1×3-in. cast-iron plates. Theoretical computations indicated that the procedures and techniques developed from small-scale laboratory studies were equally applicable to water distribution mains. To determine the validity of this hypothesis, full-scale field tests were undertaken with a 6-in. cast-iron water line extending from one of the campus wells at Michigan State University to a reservoir approximately 1,200 ft away.

To date, six field trials have been made with the 6-in. line under carefully controlled conditions. All trials have been remarkably successful in coating a 2-ft section of 6-in. pipe which was set on the end of the 1,200-ft line as it discharged into the reservoir.

Cleaning of Pipe

Because of the success with the field trials, there is now good reason to believe that a useful, practical method has been devised for arresting corrosion in cast-iron and steel water mains. When properly developed, the calcite coatings are thin, dense, and relatively hard. The ability of the coatings to prevent and arrest corrosion is excellent. When the coated pipe carries water supersaturated with calcium carbonate—that is, a water with a positive saturation or Langelier index—the coating improves with time and increases in both thickness and density. It is, of course, necessary to recognize that the same forces that initially deposit the coatings will remove them if the water is undersaturated with calcium carbonate. The use of these calcite coatings is therefore restricted to waters that have been, or will be, adjusted to a positive saturation index by the use of lime, soda ash, or caustic soda.

Before coating, the main must be free from soft deposits of rust or other material that prevents the development of a strong bond between the pipe wall and the calcite deposition. With badly corroded lines that have had a serious reduction in carrying capacity, the easiest and best procedure is to clean

the mains by conventional techniques. With lines that have not deteriorated badly, it seems probable that a satisfactory bond can be obtained by the addition of 2 ppm or more polyphosphate to the water for a period of several weeks. Following this, the water should be adjusted to a momentary excess (ME) of calcium carbonate of 12-15 ppm by procedures to be described later. If this ME level is maintained while the deposition of calcium carbonate is controlled with approximately 0.25 ppm polyphosphate, the rate of corrosion can probably be reduced to a level that permits the removal of soft deposits of rust and other material by conventional hydrant flushing. As a final step, it may be necessary to apply the given procedures on one or more occasions to completely arrest corrosion.

Coating Development

Development of the thin, dense calcite coating, which has proved so effective in experiments with Michigan State University well water, is by the use of a proper level of calcium carbonate supersaturation in the presence of 0.5 ppm polyphosphate. For proper coating, a displacement velocity of 4 fps or more in the pipe is desirable. The higher the velocity, the better the coating.

The required level of calcium carbonate supersaturation for treatment seems to be best stated in terms of the value that the author calls the driving force index (DFI). The DFI is the product of the calcium concentration and the carbonate (CO_3^{--}) concentration, divided by the solubility product constant of calcium carbonate, corrected for dissolved solids and temperature. The calcium and carbonate

concentrations are expressed in parts per million of calcium carbonate. For convenience, the conventional solubility product constant of calcium carbonate, K_s' , is also expressed in parts per million and is, therefore, stated as $K_s' \times 10^{10}$.

It has been the author's experience that a DFI of 150-200 is required for depositing the glasslike coatings that provide the best anticorrosion protection. Until more field data are available from various waters, it is suggested that a DFI of 200 be selected for initial trials. Fortunately, the process is not dependent on this theoretical value for control, and the suggested DFI value is used only for the purpose of establishing the proper pH for beginning the coating procedures. In practice, the coating is controlled by maintaining a calcium drop of approximately 2.5 ppm per inch of pipe diameter per 1,000 ft of pipe.

Simplified Procedures

In preparing procedures to carry out techniques, the author was aware that the terms driving force index, momentary excess, solubility product constant, and carbonate ion concentration are not familiar to many water utility personnel who may find real use for the techniques developed. Therefore, the procedures are written for persons in the field who are lacking in specialized knowledge. Directions have been prepared for detailed procedures that can be followed with good success by personnel whose technical laboratory experience is limited to the preparation of simple standard solutions, titration for alkalinity and hardness, and measurement of pH with conventional pH meters. Steps for these procedures are:

1. A simple standard solution of sodium carbonate or sodium hydroxide is prepared. A 1-liter sample of tap water from the distribution system is then treated with polyphosphate and titrated with the standard solution to define a pH-alkalinity curve for the distribution system water.

2. Chemical feed rates are computed for maintaining the calcium, pH, and

4. The pH of the water is finally adjusted to cause a calcium drop of 2.5 ppm per inch of pipe diameter per 1,000 ft of the line undergoing treatment. After 2 hr, a hard, dense coating should result.

For pH adjustment, either sodium hydroxide or soda ash may be used. With waters of 150 ppm alkalinity (as CaCO_3) or more, the use of sodium

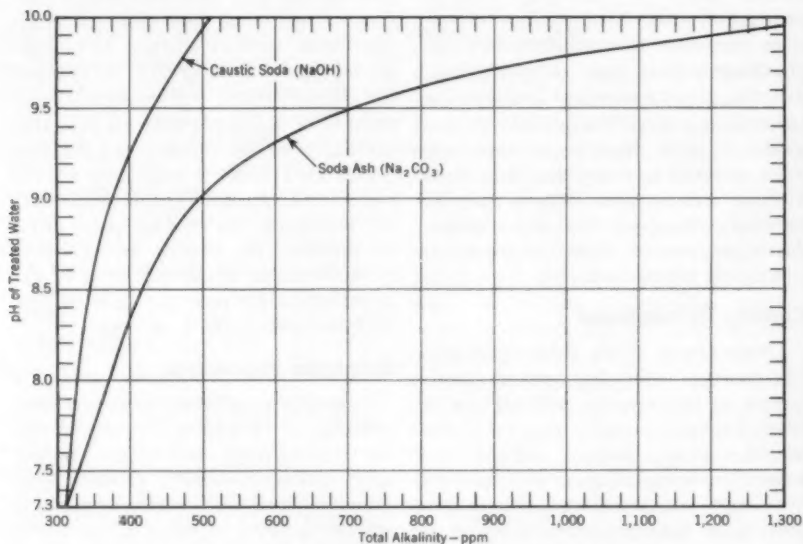


Fig. 1. pH-Alkalinity Curves for Michigan State University Tap Water

Total alkalinity values in the abscissa represent the initial plus the added alkalinities. The tap water had an initial pH of 7.3 and an initial alkalinity of 312 ppm.

polyphosphate levels necessary for laying down the coating.

3. Polyphosphate is fed to establish the required concentration of 0.5 ppm in the distribution system. If the calcium concentration is less than 150 ppm, calcium chloride is fed as required. Sodium carbonate or sodium hydroxide is then added to establish the required initial pH level.

hydroxide or caustic soda is probably most effective. With waters of less than 150 ppm alkalinity, soda ash should be used.

Applications

A line 1,200 ft long has been successfully coated, and there seems no reason to doubt that mains a mile or more in length can be coated at one

time. Because the pH of the water undergoing treatment is quite high, it is not satisfactory for domestic or industrial use, but the water can be used if it is discharged into a reservoir or neutralized by mixing with water in the distribution system. If there is no objection to wasting the treated water, it may be discharged through one or more fire hydrants during the 2 hr of treatment. Chemical feed is most easily accomplished through corporation cocks in the mains. Details on feed equipment are given in a recent JOURNAL article.¹

Because of the requirements of a pipe velocity of 4 fps or more, it may be necessary to increase the flow in the main undergoing treatment by adjusting valves of the system. The author has found displacement velocity to be most easily measured by initiating the chemical feed for pH adjustment at a given "zero" time and measuring the time for the pH change to reach the end of the line that is to be coated. If wide fluctuations in the flow are anticipated, it will probably be necessary to open one or more fire hydrants to insure a constant velocity, polyphosphate concentration, and pH level.

Costs

The cost of chemicals for the coating process is small, perhaps \$5-\$10 per mile of pipe treated. A more important cost is that for maintaining a polyphosphate concentration of 0.25 ppm and a ME of 12-15 ppm in the distribution system on a permanent basis. This cost will vary with different waters and will depend on the calcium level, alkalinity, and carbon dioxide content. For lime-softened waters, the cost is negligible. With Michigan State University water, which contains

220 ppm calcium, 325 ppm alkalinity (both as CaCO_3), and an initial pH of 7.3, the cost of maintaining 0.25 ppm polyphosphate and adjusting the pH level to 8.6 for a ME of approximately 14 ppm is estimated at \$6.26 per 1,000,000 gal, when caustic soda at a cost of 7 cents per pound is used. The cost of polyphosphate is about 20 cents a pound. For a water of low calcium hardness and alkalinity, the cost is estimated to be \$10 per

TABLE 1

Flows and Chemical Costs Required to Maintain a Displacement Velocity of Approximately 4 fps

Pipe Size in.	Flow Rate gpm	Flow Velocity fps	Water Treated in 2 hr gal	Cost of Chem- icals* \$
6	350	3.89	42,000	6.20
8	600	3.85	72,000	10.85
10	1,000	4.07	120,000	18.00
12	1,400	3.98	160,000	25.39
14	1,800	4.19	196,000	29.50
16	2,200	3.86	264,000	39.80
20	3,500	3.89	420,000	63.30
24	6,000	4.25	720,000	108.00
30	9,000	4.10	1,080,000	162.50

* Cost of raising the DFI to 200 and feeding 0.50 ppm polyphosphate for 2 hr.

1,000,000 gal, when soda ash at a cost of 5 cents per pound is used. With lime, water with these same characteristics can be treated to the proper level for pH and polyphosphate for about \$1 per 1,000,000 gal, but the calcium hardness would probably be increased to 50-70 ppm.

The cost of \$6.26 per 1,000,000 gal for the coating process used with Michigan State University water was calculated in this manner: For the required pH of 8.6, the alkalinity is 350 ppm (Fig. 1). This requires the addi-

tion of 350 ppm minus 325 ppm, or 25 ppm alkalinity. The weight of 1 gal of water is 8.34 lb. The molecular weight of NaOH is 40; of CaCO_3 , 100. The cost of caustic soda per million gallons is:

$$8.34 \times 25 \times \frac{40}{100} \times 7 \text{ cents} = \$5.84.$$

The cost of polyphosphate per million gallons is:

$$8.34 \times 0.25 \times 20 \text{ cents} = 42 \text{ cents}.$$

Thus, the total cost per million gallons is:

$$\$5.84 + 42 \text{ cents} = \$6.26.$$

Table 1 shows the flows and chemical costs required for pipe of various size to maintain a displacement velocity of approximately 4 fps in Michigan State University water. Chemical costs in Table 1 are for the 2-hr coating process only, and are not related to the



Fig. 2. Nomograph for Evaluation of Corrected Solubility Product for Calcium Carbonate

When the temperature and the concentration of dissolved solids are known, the corrected solubility product for calcium carbonate (K_s') can be determined by extending to the scale on the right the line connecting the two known values. The nomograph is based on an equation given by Larson and Buswell³:

$$pK_s' = pK_s - \frac{4\sqrt{\mu}}{1 + 3.9\sqrt{\mu}}$$

in which pK_s' is the negative log of the solubility product constant of calcium carbonate corrected for temperature and dissolved solids, and pK_s , the negative log of the solubility product constant of calcium carbonate.

foregoing computations for maintaining a 12-14-ppm ME.

pH-Alkalinity Curve

As mentioned before, the author has attempted to present his procedures by using detailed, simple instructions for the benefit of water supply personnel who have had no special training or background on this subject. The procedures that follow, designed to simplify the computations required for use of the techniques just described, pertain to the construction of a pH-alkalinity curve for any given water, determination of pH-alkalinity relationships for the water to be used in the coating process, preparation of chemical solutions, and the actual coating of the pipe.

The construction of a pH-alkalinity curve involves these steps:

1. Determine the total alkalinity of the tap water of the distribution system.

2. Weigh out approximately 1 g sodium hexametaphosphate ($\text{Na}_6[\text{PO}_3]_6$),* and dissolve in 100 ml of tap water. Add 1 ml of this solution to 1 liter of tap water. The resulting concentration of approximately 10 ppm polyphosphate in the tap water prevents precipitation of calcium carbonate in the next steps.

3. Both soda ash (Na_2CO_3) and caustic soda (NaOH) may be used for these procedures. With soda ash, dissolve 10.6 g in 1 liter of distilled or demineralized water. Fill a buret with this standard Na_2CO_3 solution (10 mg/ml as CaCO_3) and add small increments of the solution to the 1-liter

sample of phosphate-treated water from Step 2, checking the pH after each addition. For every milliliter of standard solution added, the alkalinity of the water is increased almost 10 ppm (there is some dilution). For a check on alkalinity, titrate the final sample with standard acid, correcting for dilution. With caustic soda, proceed in an identical manner, using 4.0 g NaOH dissolved in 1 liter of distilled or demineralized water.

4. Plot a pH-alkalinity curve as shown in Fig. 1. For this curve, total alkalinity is equal to the alkalinity of the untreated water plus the added alkalinity from the standard Na_2CO_3 or NaOH solution. The values of added alkalinity must be corrected for dilution, with the use of the formula:

Total alkalinity

= initial alkalinity of tap water

$$+ \frac{\text{ml standard solution added} \times 10,000}{1,000 + \text{ml standard solution added}}$$

Figure 1 shows plots of Michigan State University tap water when the above procedures were used with soda ash and caustic soda.

pH-Alkalinity Relationships

For a determination of the pH-alkalinity relationships for the coating operation, the values for total solids, $K_a' \times 10^{10}$, and CO_3^{--} must first be computed.

The concentration of total dissolved solids of the water during the 2-hr process is equal to the concentration of dissolved solids in the untreated tap water plus the concentration of dissolved solids from the alkalinity and calcium (if added) that is fed to the water during the coating operation. For computation of dissolved-solids concentration, these steps should be followed:

* Calgon, made by Calgon, Inc., Pittsburgh, Pa., was used in Michigan State University experiments. Other polyphosphates serve equally well, but weight relationships vary slightly from those used here.

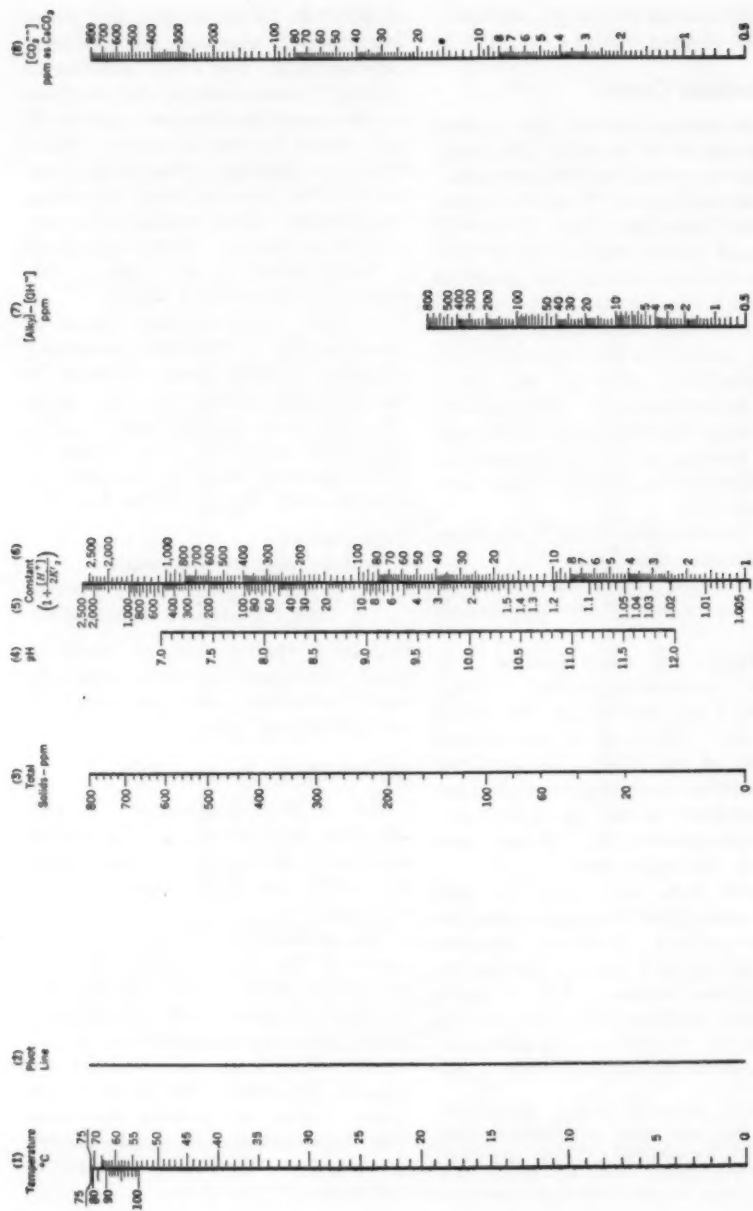


Fig. 3. Nomograph for Evaluation of Carbonate Alkalinity

To use: (1) align temperature of pH determination and total solids; (2) pivot on Line 2 to proper pH, and read constant on Scale 5; (3) align constant on Scale 6 with value for nonhydroxide alkalinity on Scale 7, and read carbonate alkalinity on Scale 8.

1. Assume a pH of 9.4 for the coating operation, and determine the corresponding alkalinity from the pH-alkalinity curve prepared before.

2. Subtract the alkalinity in the untreated tap water from the alkalinity value from the curve. The resulting value is a *first trial* in determining the alkalinity that must be added to give a pH level that will cause the proper coating development.

3. If the calcium concentration (as CaCO_3) of the water is 150 ppm or more, no correction for calcium is required in the next step. If the calcium concentration is less than 150 ppm, subtract the calcium concentration of the tap water from 150 to determine the concentration of calcium that must be added.

4. Using an approximate relationship that 1 ppm alkalinity and 1 ppm calcium (both as CaCO_3) equal 1 ppm dissolved solids, add the values for increased alkalinity from Step 2, and add the values for any increase of calcium from Step 3, to the normal dissolved-solids concentration of the water. The resulting sum is the value to be used for the dissolved-solids concentration in computations for K_s' and CO_3^{--} .

5. Determine $K_s' \times 10^{10}$ from the nomograph in Fig. 2, using the concentration of total dissolved solids from Step 4. The temperature value for use with this nomograph and with the nomograph on CO_3^{--} evaluation (Fig. 3) is that of the distribution system tap water.

6. Determine the required CO_3^{--} concentration for the coating process. This computation is made by assuming a DFI of 200 in the relationship:

$$\text{DFI} = \frac{\text{Ca}^{++} \times \text{CO}_3^{--}}{K_s' \times 10^{10}}$$

For the calcium (Ca^{++}) value of this relationship, use the calcium concentration (as CaCO_3) of the tap water if it is 150 ppm or more. If the calcium concentration is less than 150 ppm, use a calcium concentration of 150 ppm in these computations.

7. From the nomograph in Fig. 3, and with the use of the temperature of the distribution system tap water, the concentration of total dissolved solids found in Steps 1-4, and the CO_3^{--} level from Step 6, find the correct pH and alkalinity by trial and error. For example, with Michigan State University tap water, computations were made with these data: normal calcium level (as CaCO_3), 221 ppm; temperature of tap water, 13°C ; normal alkalinity (as CaCO_3), 315 ppm; normal total dissolved solids, 325 ppm; and alkalinity (as CaCO_3), when soda ash is used to adjust to pH 9.4, 650 ppm, from Fig. 1. Then:

$$a. \text{ Added alkalinity} = 650 - 315 = 335 \text{ ppm}$$

$$b. \text{ Total dissolved solids} = 325 + 335 = 660 \text{ ppm}$$

$$c. K_s' \times 10^{10} = 140 \text{ (from Fig. 2)}$$

$$d. 200 = \frac{221 \times \text{CO}_3^{--}}{140}$$

$$e. \text{ Required } \text{CO}_3^{--} = \frac{140 \times 200}{221} = 127 \text{ ppm.}$$

For pH 9.4, the concentration of CO_3^{--} , from Fig. 3, is 125 ppm. This is close enough, and the assumed pH of 9.4 is correct.

For an example of the procedure that results when the assumed pH of 9.4 is not correct, identical conditions may be assumed with a calcium concentration of 150 ppm instead of the concentration of 221 ppm. Then:

$$a. \text{ Required } \text{CO}_3^{--} = \frac{140 \times 200}{150} = 187 \text{ ppm}$$

b. For pH 9.40, the CO_3^{--} concentration from Fig. 3 is 125 ppm (try pH level of 9.70)

c. For pH 9.70, the CO_3^{--} concentration from Fig. 3 is 210 ppm (try pH level 9.60)

d. For pH 9.60, the CO_3^{--} concentration from Fig. 3 is 180 ppm (close enough to required value of 187 ppm).

Therefore, for these conditions, a pH level of 9.60 is almost correct. It is now well to return to Step 4 for a recomputation of $K_s' \times 10^{10}$ on a basis of 760 ppm dissolved solids, instead of the 660 ppm used in computations with Michigan State University tap water. K_s' from Fig. 2 is then 150, and the required concentration of CO_3^{--} is $\frac{150 \times 200}{150} = 200$ ppm.

For this, the pH level of 9.7 is almost correct and should be used.

8. If the calcium concentration (as CaCO_3) of the water is less than 150 ppm, additional calcium must be added during the 2-hr coating process. The most convenient source of the additional calcium is calcium chloride (CaCl_2). Sample calculations for determining the required feed rate may be made with these data: solubility of calcium chloride, 40 lb per 100 lb of water; flow, 400 gpm; time for coating, 2 hr; time for feed adjustment, 0.5 hr; and calcium concentration (as CaCO_3) in tap water, 40 ppm. Then:

a. 150 ppm (required calcium concentration) - 40 = 110 ppm calcium (as CaCO_3) to be added

b. $400 \text{ gpm} \times 150 \text{ min} \times 8.34 \text{ lb/gal} = 500,400 \text{ lb}$ of water that flows in the main during treatment

c. 110 ppm calcium requires $110 \times \frac{111}{100} = 122$ ppm calcium chloride

d. Calcium chloride required = $\frac{500,400}{1,000,000} \times 122 = 62 \text{ lb}$.

For convenience, dissolve the 62 lb of calcium chloride in a 50-gal drum of water. Then:

e. Feed rate = $\frac{50 \text{ gal}}{150 \text{ min}} = 0.33 \text{ gpm}$.

Therefore, dissolve 62 lb of calcium chloride in 50 gal of water, and feed at a rate of 1 gal every 3 min.

Chemical Solutions

1. Calculate the amount of metaphosphate needed for the 2-hr process of coating development. Sample computations may be made with these data: flow in main, 400 gpm; time of run, 2 hr; time for adjustment, 2 hr; and phosphate concentration (as PO_4) desired, 0.5 ppm. Use sodium hexametaphosphate, which is approximately 100 per cent pure as PO_4 . Then:

a. $400 \text{ gpm} \times 240 \text{ min} \times 8.34 \text{ lb/gal} = 800,640 \text{ lb}$ of water that flows in the main during treatment

b. An addition of 0.5 lb of hexametaphosphate per million pounds of water (0.5 ppm) requires:

$$\frac{800,640}{1,000,000} \times 0.5 = 0.4 \text{ lb}$$

c. Feed rate = $50 \text{ gal/4 hr} = 12.5 \text{ gph} = 0.21 \text{ gpm}$.

Therefore, for a flow of 400 gpm, 0.4 lb of hexametaphosphate should be dissolved in 50 gal of water and fed continuously at a rate of 12.5 gph for the required 4-hr period. The resulting polyphosphate concentration (as PO_4) is 0.5 ppm.

2. Calculate the amount of soda ash or caustic soda required and the feed rate required for proper pH adjustments. Sample computations may be made with these data: soda ash, approximately 100 per cent pure; flow in water main, 400 gpm; alkalinity (as CaCO_3) of untreated water, 325 ppm; alkalinity (as CaCO_3) desired

for 2-hr coating process, 650 ppm (from Step 2, under "pH-Alkalinity Relationships"); and time of run, 2 hr, plus 30 min extra, for a total of 150 min. Then:

a. Alkalinity to be added = 650 - 325 = 325 ppm

b. $400 \text{ gpm} \times 150 \text{ min} \times 8.34 \text{ lb/gal} = 500,400 \text{ lb}$ of water that flows in the main during treatment

c. Soda ash to be added = 325 $\times 1.06 = 345 \text{ ppm}$

d. Amount of soda ash required for treatment is:

$$\frac{500,400 \text{ lb} \times 345 \text{ ppm}}{1,000,000} = 175 \text{ lb}$$

e. As 44 lb of soda ash dissolves easily in 100 gal of water, the quantity of feed solution needed is:

$$\frac{100 \times 175}{44} = 400 \text{ gal}$$

f. Feed rate = 400 gal in 150 min = 2.66 gpm.

3. With caustic soda, 420 ppm alkalinity, instead of the 650 ppm used with soda ash, is required for a pH level of 9.4. Therefore, computations of the type shown first in Step 7 under "pH-Alkalinity Relationships" will be required. These computations show that 470 ppm alkalinity is required to raise the pH to a level of 9.5 for the required DFI of 200. Then:

a. Alkalinity to be added = 470 - 325 = 145 ppm (Fig. 1)

b. 500,400 lb of water flows in the main during 150 min of treatment

c. Caustic soda to be added = 145 $\times 0.40 = 58 \text{ ppm}$

d. Amount of caustic soda required for treatment is:

$$\frac{500,400 \text{ lb} \times 58 \text{ ppm}}{1,000,000} = 29 \text{ lb}$$

e. The 29 lb of caustic soda dissolves easily in 50 gal of water, and the rate of feed is:

$$\frac{50 \text{ gal}}{150 \text{ min}} = 0.33 \text{ gpm.}$$

It is probably more convenient to dissolve the caustic soda in 100 gal of water and feed at a rate of 0.66 gpm.

Coating Process

1. For 2 hr before starting the 2-hr coating development process, add sodium hexametaphosphate (or any other polyphosphate) to the tap water in order to fix the phosphate concentration (as PO_4) at approximately 0.5 ppm. It is very important that the polyphosphate concentration be in this approximate range, and it is absolutely necessary that it be thoroughly mixed with the water before caustic soda or sodium carbonate is added for pH adjustment. Moreover, the polyphosphate feed must be constant throughout the entire run. A variation of more than 0.05 ppm in polyphosphate concentration may result in an unsatisfactory coating.

2. Once the polyphosphate concentration has been stabilized at a concentration of approximately 0.50 ppm, begin the addition of sodium carbonate or sodium hydroxide at the calculated flow rate. Using ethylenediaminetetraacetic tetrasodium salt (EDTA),* check the calcium level of the water upstream from the point of pH adjustment every 30 min or oftener during the coating process. Continuously check the calcium level of the water at a point 1,000 ft or farther downstream from the point of pH adjustment for the first 20 min of the 2-hr period. A drop in concentration of calcium (as CaCO_3) of approximately 2.5 ppm per 1,000 ft of pipe per inch of pipe diameter should be observed

* Hach Versenate and Calver, made by Dow Chemical Co., Midland, Mich., were used in these studies.

within 15–20 min after pH adjustment is started.

3. If the drop in calcium concentration is less than 2.5 ppm per inch of diameter per 1,000 ft of pipe, increase the pH level by feeding more NaOH or Na_2CO_3 until this drop is observed. If the drop is more than 2.5 ppm, reduce the chemical feed for pH adjustment until the required level is reached. Then, for the entire 2-hr period, check the calcium and pH level at the downstream point every 10 min or oftener.

4. Maintain the 2.5-ppm calcium drop for the first hour of the run, at which time it may be increased to 3.5 ppm per inch of pipe diameter per 1,000 ft, but no greater. During the first 60–90 min of the treatment process, a sudden drop in calcium concentration at the downstream point may be noted. If this occurs, it is necessary to reduce immediately the feed rate of the caustic soda or sodium carbonate to a level that results in the required drop in calcium concentration.

5. When a large increase in calcium drop is experienced during the run, the reduced calcium concentration at the downstream check point is accompanied by a drop in pH. Decreasing the chemical feed rate to the required value usually results in an increase in both pH and calcium concentration at the downstream point. Control of the process must, therefore, be maintained by a continued determination of the calcium loss between the two check points. Changes in pH level between the two points are useful for guidance

during the period of coating development but are an inadequate substitute for a continued check on the calcium level.

6. At the end of the 2-hr period, the main should be coated with a thin, dense, calcium carbonate coating that should largely arrest a loss of carrying capacity and the development of deposits of flocculent rust in the pipe.

Acknowledgments

The author wishes to express his appreciation to John F. Dye,⁴ superintendent of the Lansing (Mich.) water treatment plant, for development of Fig. 2 and 3. Appreciation is also expressed for the assistance of Kenneth B. Yerrick, graduate in chemistry, and Cordell E. Johnson, Roger O. Gatlin, and William J. Hahn, undergraduates in the College of Engineering, all of Michigan State University, who assisted in the preparation of the procedures.

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Standardization of Design Procedures for Prestressed-Concrete Pipe

—Robert E. Bald—

A paper presented on Jun. 16, 1960, at the Pennsylvania Section Meeting, Pittsburgh, Pa., by Robert E. Bald, Chief Engr., Lock Joint Pipe Co., East Orange, N.J., and Chairman, Specifications and Technical Problems Committee, American Concrete Pressure Pipe Assn.

AFTER several years of study and review, the Specifications and Technical Problem Committee of the American Concrete Pressure Pipe Association has developed a design procedure for prestressed pipe which has been adopted by the industry.

If there were no inelastic losses to consider in the design of prestressed pipe, there would be few differences between the design procedures used by the various manufacturers of this type of pipe, but inelastic losses do occur in the high-tensile wire and in the concrete. These losses must be determined empirically and must be defined in terms that permit application to a wide range of diameters, pressures, and external load conditions with reasonable results. Because different methods of determining and defining these losses have been used by various producers, their design procedures have differed also. Consequently, confusion sometimes resulted when an engineer or purchaser received slightly different designs resulting from considerably different design procedures, but all based on the same conditions of internal pressure and external load.

Description of Pipe

AWWA C301¹ establishes design requirements for two types of prestressed-concrete cylinder pipe, lined-

cylinder pipe and embedded-cylinder pipe. Lined-cylinder pipe (Fig. 1) is confined to 16–48 in. diameters. The pipe core consists of a light-gage steel cylinder, with steel joint rings attached to its ends, lined with high-strength concrete. The high-tensile wire is wrapped around this core at a carefully controlled spacing and measured tension. A coating machine applies a dense premixed mortar coating around and over the steel cylinder and wire. Embedded-cylinder pipe (Fig. 2) is similar to lined-cylinder pipe, except that the steel cylinder is encased in the concrete core. Consequently, the wire is wrapped directly on the concrete. The prestressed cores of larger sizes of embedded-cylinder pipe are often encased in a vertically cast concrete coating rather than mortar. AWWA C301 specifically covers embedded-cylinder pipe of 24–72-in. diameter, but it permits larger diameters.

Historical Background

Prestressed-concrete cylinder pipe has been produced commercially since 1942. In 1952, prestressed-concrete embedded-cylinder pipe was introduced. Several thousand miles of these types of pipe, from 16 to 120 in. in diameter, are in service.

Originally pipe was designed so that the elastic limits of the prestressing wire and the steel cylinder were reached simultaneously at a pressure $2\frac{1}{2}$ times the operating pressure. The gross-wrapping stress was 40–50 per cent of the ultimate strength of the wire, including a 20 per cent allowance for inelastic losses in the wire and concrete.

Subsequently, a higher gross-wrapping stress was adopted, in keeping with an international trend in both lined and circular prestressed structures. Inelastic losses were defined in terms of initial elastic strain by some

cally, it is only necessary to know their areas and elastic moduli together with the wire wrapping stress in order to compute the theoretical elastic strain or deformation immediately after prestressing.

Based upon extensive test results for centrifugally placed concrete an initial modulus of elasticity of 4.7×10^6 psi and a final, or resultant modulus of 5.6×10^6 psi have been established for design. Similarly, for high-strength vibrated concrete, an initial modulus of 4.0×10^6 psi and a resultant modulus of 4.7×10^6 are suitable for design. For practical purposes, the wire

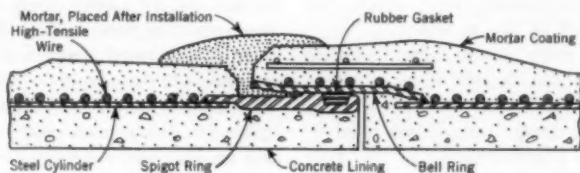


Fig. 1. Lined-Cylinder Prestressed-Concrete Pipe and Joint

The cylinder is lined centrifugally with dense concrete and high-tensile wire is wrapped around the steel cylinder. The wrapped core is then covered with a mortar coating. Joint mortar, of course, is placed after installation.

manufacturers, with the various losses lumped together as a single factor. The factor was a function of the core thickness and compression steel area. Others separated the losses for each of the components and used constant factors. The concrete inelastic strain was defined in terms of residual elastic strain.

Elastic Strains

As the high tensile wire is wrapped on the pipe core, equilibrium must be maintained between the force exerted by the core and that of the wire. Because the materials are acting elasti-

and steel cylinder are assumed to have an elastic modulus of 28×10^6 psi. These values result in modular ratios (n_i) of 6 for centrifugal concrete and 7 for vibrated concrete for the initial condition and (n_r) of 5 and 6 for the resultant condition.

Inelastic Strains

Once the core has been prestressed, wire relaxation and concrete creep begin occurring in both lined-cylinder pipe and embedded-cylinder pipe. In the latter, embedment of the wires into the concrete core also must be considered. Shrinkage in a properly cured

high-strength concrete core is of little significance, as most of it will occur before stressing of the core. What little shrinkage may occur after prestressing will be accounted for in the allowance for creep in the concrete, which has been determined by tests on production-made pipe.

Wire Relaxation

Wire relaxation is the inelastic stretching of steel wire under sustained tensile load. The magnitude of this relaxation varies with the type of wire and increases in proportion to

wrapping stress for pipe does not exceed 70 per cent of the average ultimate tensile strength of the wire and has been set at 73 per cent of the required minimum ultimate tensile strength (140,000 psi for No. 6 gage wire). This stress is instantaneously reduced by the elastic deformation of the core. The reduction varies with the pipe class and wall thickness but is seldom less than 10,000 psi. Therefore the wire stress producing relaxation will normally be less than 65 per cent of its average ultimate strength, for which Fig. 3 indicates a stress reduction of

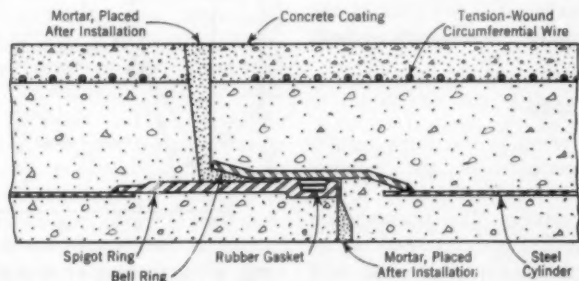


Fig. 2. Embedded-Cylinder Prestressed-Concrete Pipe and Joint

In this type of pipe, the tension-wound wire is wrapped around a concrete core in which the steel cylinder is embedded, rather than around the steel itself.

the stress. Much research on this subject has been done by wire producers here and abroad. For the cold-drawn wire used on prestressed pipe, relaxation approaches an asymptotic limit in about 1 month. Figure 3 is reproduced from a paper by W. O. Everling, director of research at American Steel and Wire Division of US Steel Corporation. It shows the percentage of relaxation for cold-drawn and stress-relieved high-tensile wire held at various constant lengths for 1,000 hr (approximately 40 days). The

6.4 per cent. Cold working of the wire under tension during the prestressing operation and the fact that the wire is not held at constant length as it relaxes on the pipe core so reduce this loss that 5 per cent of the gross wrapping stress is a conservative allowance.

In lined-cylinder pipe the high-tensile wire is wrapped on the steel cylinder, which distributes the load into the concrete core. As a result there is no significant local yielding of the concrete beneath the wires. In embedded-cylinder pipe, on the other

hand, the wires are in direct contact with the concrete core. Initially an extremely high stress exists in the concrete owing to the small surface in contact with the wire. As the wire indents the concrete, the supporting surface increases until equilibrium is reached. This takes place almost instantaneously as the wire is wrapped onto the core. No further embedment of the wire will occur, for while the concrete is gaining strength with age, the stress in the wire is being reduced by inelastic strains in the wire and concrete. Therefore, this reduction is also treated as an initial loss in wire stress. A value of 5 per cent of the gross wrapping stress has been assumed.

Concrete Creep

Concrete creep is the most significant and complex of the inelastic strains or losses. Since early in the twentieth century, this phenomenon has been studied and reported on by many recognized experts in the field of concrete. Most of the published material relates to tests on unrestrained samples of concrete subjected to a constant axial load. Consequently, many of the quantitative data are not applicable to the concrete restrained in a pipe core resisting a radial load that gradually reduces because of the inelastic deformation of the core and is reduced considerably further when hydrostatic pressure is applied to the pipeline. Nevertheless, most of the qualitative information is useful in determining the factors influencing creep in concrete. Some of the principal conclusions are:

1. Creep in concrete under constant load approaches an asymptotic limit after 2-5 years.

2. Creep strain is reversible, within limits, when the load is removed or appreciably reduced. The amount of recovery varies inversely with the length of time the load has been applied.

3. Creep varies inversely with the strength of the concrete at the time of loading.

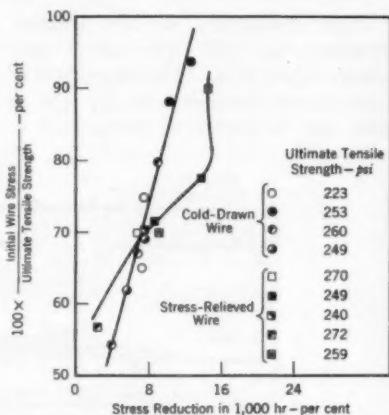


Fig. 3. Relaxation of Cold-Drawn and Stress-Relieved High-Tensile Wires

Relaxation data were determined for various sections of wire held at constant length for 1,000 hr after initial stressing to various percentages of ultimate tensile strength.

4. Creep approaches direct proportionality to stress within the elastic range of concrete.

5. Creep is influenced by the water-cement ratio, the type of cement, the richness of mix, aggregate size and gradation, type of aggregate and size of specimen tested.

6. Creep is noticeably affected by the moisture conditions under which the concrete is stored. Normal temperature variations show little effect,

assuming a constant relative humidity is maintained.

Because there are so many variables involved, it is difficult to establish accurate values for concrete creep in prestressed-concrete pipe. It is generally agreed that long-term hydrostatic testing of various diameters and classes of pipe is the best solution. By periodic hydrostatic testing of a pipe, with strain gages attached to the high-tensile wires and to the steel cylinder, one can determine the pressure at which the compression in the concrete is dissipated. This pressure, called the "zero concrete stress pressure," reduces in larger increments during early stages of testing and in smaller and smaller increments as time goes on. Eventually, in 2-5 years, this pressure stabilizes. The maximum creep for the concrete in a pipe can be approximated by computing the strain in the concrete immediately after prestressing and again once the zero concrete stress pressure has stabilized. After allowance for wire relaxation and wire embedment, the difference between the two computations represents the maximum concrete creep strain for the pipe. As most pipe is put in service within a year after manufacture, the actual creep strain would be considerably less for a number of reasons. A major part of the compression induced by the prestressing wires is relieved by the internal water pressure. As noted above, there will be some recovery of creep strains owing to the reduced load, and the smaller compression will produce a smaller ultimate creep strain. In addition, the concrete will be fully saturated, in contrast to test specimens, which are stored under relatively dry conditions. The moist environment

causes a further gain in concrete strength.

Although it is obviously a conservative value, the maximum creep strain, as determined by hydrostatic test, is used in establishing the creep factor for design. For the design procedure presented below the creep factor (C_r) has been defined in relation to the residual elastic strain. Values of 2.0 for vibrated concrete in embedded-cylinder pipe and 1.5 for centrifugally placed concrete in lined-cylinder pipe have been shown to be extremely conservative for normal production pipe, based on a large number of tests.

Design Procedure

Formulas can be derived for the stress in the wire, cylinder, and concrete or for pressure in the pipe for any condition desired. Several of these, however, are of primary importance. These are the compression in the concrete after all losses, referred to as the resultant compression, f_{cr} ; the zero concrete stress pressure, P_0 ; and the pressure at which the wire is stressed to the wrapping stress, or the wire elastic-limit pressure, P_1 . Of secondary interest are the initial concrete stress, f_{ci} ; the initial and resultant wire stresses, f_{si} and f_{sr} ; the resultant cylinder stress, f_{yr} ; and the bursting pressure of the pipe, P_b .

For simplicity of presentation, the initial condition just after prestressing is considered first. It is assumed that the wire and embedment losses have occurred, but no concrete creep losses. Balancing of forces yields the expression

$$f_{si}A_s = f_{ci}A_c + f_{yi}A_y$$

in which A_s is the wire area, A_c is the concrete area, and A_y is the cylin-

der area. As the initial wire stress f_{si} equals the wrapping stress f_{wg} less the sum of the elastic strain $n_1 f_{ei}$, the wire relaxation loss $R_1 f_{wg}$ and the embedment loss $R_2 f_{wg}$, it can be seen that

$$f_{si} = f_{wg}(1 - R_1 - R_2) - n_1 f_{ei}$$

and

$$f_{ei} = \frac{A_s f_{wg}(1 - R_1 - R_2)}{A_c + n_1(A_s + A_y)}$$

The resultant concrete stress f_{cr} is that which exists after full inelastic creep strain has occurred in the concrete. As the concrete shortens inelastically, the wire and cylinder shorten elastically, causing a decrease in wire stress and an increase in cylinder stress. The reduced wire stress also causes an elastic loss of strain in the concrete. As the mathematics is somewhat cumbersome, only the final equation for f_{cr} is presented:

$$f_{cr} = \frac{A_s f_{wg}(1 - R_1 - R_2)}{A_c + n_1(A_s + A_y)} \times \frac{A_c + n_r(A_s + A_y)}{A_c + n_r(A_s + A_y)(1 + C_r)}$$

The resultant stresses in the wire and cylinder can be most simply stated in terms of the stresses already defined.

$$f_{wr} = f_{si} - n_r[f_{cr}(1 + C_r) - f_{ei}]$$

$$f_{yr} = n_1 f_{ei} + (f_{si} - f_{wr})$$

The zero concrete stress pressure is the pressure required to dissipate the resultant compression in the concrete core. The minimum zero concrete stress pressure has been set at 125 per cent of the design pressure. The formula for the zero concrete stress pressure is:

$$P_0 = \frac{f_{wr}}{6D_y} [A_c + n_r(A_s + A_y)]$$

in which D_y is the outside diameter of the cylinder.

The wire elastic-limit pressure occurs when the stress in the wire reaches the gross wrapping stress applied at the time of prestressing. This elastic, or proportional, limit in the wire was created by cold working at the wrapping stress. Owing to the initial wire losses which did not affect the elastic deformation of the cylinder, the cylinder reaches its point of transition from compression to tension at a wire stress of $f_{wg} - (R_1 + R_2)f_{wg}$. Therefore, the cylinder will be in tension at the wire elastic-limit pressure. AWWA C301 requires that this pressure be at least twice the design pressure. The formula for the wire elastic-limit pressure is:

$$P_1 = \frac{f_{wg}}{6D_y} [A_c + A_y(R_1 + R_2)]$$

The bursting pressure is a function of the ultimate strength of the wire and the tensile stress in the steel cylinder when the wire reaches its maximum extension. It has been determined that the minimum stress in the restrained cylinder, f_{ybh} , is approximately 41,000 psi for lined-cylinder pipe and 45,000 psi for embedded-cylinder pipe. The formula for bursting pressure is:

$$P_b = \frac{A_s f_{su} + A_y f_{ybh}}{6D_y}$$

Because the minimum ultimate strength of the wire is used for f_{su} , actual bursting pressures are usually much greater than the computed bursting pressure.

Conclusion

By using the design procedure presented above, engineers will now be able to check the designs submitted by any of the producers of pipe made

under C301. There will be some variations between old and new designs, but these will be relatively small. In some cases the wire area will be increased and in others it will be decreased.

This procedure, together with the requirements in sec. 3.2 of AWWA C301, gives designs suitable for the internal pressure and at least 6 ft of earth cover under normal circumstances. There is also an automatic water hammer allowance of 50 psi or 40 per cent of the design pressure, whichever is greater. Therefore, most pipe can be designed according to this simple hydrostatic procedure. When

external loads exceed the equivalent of 6 ft of cover, a combined load analysis may be used.

The American Concrete Pressure Pipe Association ran a series of tests on combined loading of prestressed-concrete embedded-cylinder pipe. An article covering these tests and describing a procedure for combined load analysis will soon appear in the *Proceedings of the American Society of Civil Engineers*.

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Federal-Aid Highway Policies Affecting Utilities

James E. Kirk

A paper presented on May 5, 1960, at the Pacific Northwest Section Meeting, Portland, Ore., by James E. Kirk, Chief, Eng. Correlation Branch, Right-of-Way Div., Office of Eng., Bureau of Public Roads, Washington, D.C.

WITH highways, as with water supply, constructive planning and technologic progress have been undertaken for the overall public benefit. The continuous efforts of the nation's highway agencies have been, and will continue to be, directed toward the basic requirement and principle that highways be engineered to solve traffic problems and, at the same time, be safe and durable. This requirement for highway projects on the National System of Interstate and Defense Highways is set forth in further detail under the terms of the standards that require control of access for all sections of the interstate system. These standards were developed by the American Association of State Highway Officials (AASHO) and approved by the Bureau of Public Roads of the Department of Commerce on Jul. 7, 1956. Such access is to be controlled by the acquisition of complete access rights prior to construction, or by the construction of frontage roads, or both. These provisions were established to preserve the traffic-carrying capacity of important highways, thus warranting the large public expenditures being made for their construction, and to provide the maximum degree of safety for highway users, insofar as it can be provided through highway planning, de-

sign, right of way, construction, and operation.

Just as the national highway construction program has greatly expanded in recent years to serve the increasing demands of traffic, so has the utility industry experienced a parallel growth to meet the needs of an expanding population. Under these conditions, both the utility industry and the highway agencies have experienced an increasing difficulty in acquiring the lands necessary to accommodate their facilities, and the frequency of contact between highway and utility construction has and will continue to increase.

AASHO Utility Policy

After the passage of the 1956 Federal-Aid Highway Act, the Bureau of Public Roads of the US Department of Commerce and the state highway departments recognized that the control of access could be materially affected by the extent and manner in which publicly and privately owned utilities cross or otherwise occupy the rights of way of interstate highways. In order to carry out the intent of the law, a uniform policy was needed to establish the conditions under which the utilities could be accommodated on the rights of way of interstate highways. Many similar policies that have

been required to meet a new provision of law have been developed through the work of the committees of AASHO, with the Bureau of Public Roads assisting. Such policies developed in this manner come from the states. They are state policies and not federal standards. They are not arbitrary values forced on the states by the bureau. They represent the best thinking of highway engineers throughout the country.

Development. In recognition of the need for establishing a uniform policy on the accommodation of utilities, the AASHO Committee on Planning and Design Policies began the task of developing such a policy in the fall of 1957. After long debate and discussion, and many meetings with national segments of the utility industry, including representatives of AWWA, "A Policy on the Accommodation of Utilities on the National System of Interstate and Defense Highways" was approved by letter ballot of the states and adopted by AASHO on Jul. 30, 1959. On Sep. 30, 1959, the policy was accepted by the federal highway administrator for use on federal-aid interstate projects.

Scope. The record is well documented to show that this policy was developed cooperatively over a long period of time and represents the result of a great deal of effort by competent people from state organizations, the Bureau of Public Roads, and the utility industry. There should be no major concern regarding the soundness of the policy or its intended accomplishments. To those who may express concern over the brevity of the AASHO utility policy in respect to the increasing magnitude of the utility problem, it should be pointed out that the policy is purposely intended to be

broad in scope, as any national standard should be. In turn, it is expected that each state will develop its own policies and procedures within the framework of the AASHO policy, as may be needed to fit conditions within that state.

Provisions. The policy permits no utilities along interstate highways on new location and requires the removal of utilities that might exist along such highways where they cannot be serviced without direct access from the through-traffic roadways or ramps. It permits utilities along frontage roads and across interstate highways, all in such manner that they can be serviced without direct access from the through-traffic roadways or ramps.

The policy recognizes the inevitability of exceptions to even the most rigid rule and sets forth the conditions in an extreme case under which a utility can be located along an interstate highway on new location and where other requirements of the policy can be adjusted. "Extreme case" is just what those words imply. It will not be possible to meet the conditions set up in the policy for making an exception unless the utility costs and conditions needed to conform with the policy are quite extraordinary. On the other hand, the "extreme case" paragraph set up in the policy will enable the highway program, with respect to utilities, to be administered with recognition of the realities of a particular situation.

The approved policy insures the preservation of the principles of access control, and yet it permits the installation, adjustment, or relocation of utilities across the interstate right of way when they do not adversely affect the design, stability, traffic safety, or operation of the interstate highways. In

some instances, the retention of the existing utilities along the interstate right of way is permitted. As indicated before, it is not expected that the policy will provide an easy solution for each and every problem that may be encountered; it is hoped that good common-sense judgment will be applied where and when needed.

Application. Coordination leading to uniformity in application can best be accomplished under an operating procedure that will permit and provide for consultation and analytic study by all parties concerned. Such consultation and study should be done at the earliest possible time prior to proceeding with the details of final design of the highway and its structures. Under such an operating procedure, there is every reason to expect that a uniform application of the policy can be realized.

The ultimate attainment of any policy is dependent on a practical application of its provisions by prudent and dedicated men who never lose sight of the policy's principal objectives—in this instance, to preserve and maintain the access control of the highways of the interstate system and to increase the safety and utility of these highways to the maximum extent possible. It is expected that this can be accomplished in the best interests of the highway users and with careful consideration to insure that the interests of the utility consumer are treated reasonably and fairly.

Federal Legislation

Governing federal legislation, policies, and procedures, particularly those relating to reimbursement of costs incurred for relocating or adjusting utility facilities necessitated by highway construction, merit discussion here.

The Bureau of Public Roads is responsible for administration of the

federal-aid highway funds authorized under the federal-aid highway acts. Although there have been numerous amendments and supplemental acts, since the enactment of the first Federal-Aid Highway Act on Jul. 11, 1916, and its first two amendments of 1919 and 1921, several cardinal principles of the initial acts have not been changed. These principles are basic to all federal-aid highway undertakings.

It is important to understand that the program being discussed here is a cooperative program in which all projects involving federal-aid reimbursement are to be proposed by the state highway departments. It is a program of federal aid for the purpose of improving a limited mileage of highways jointly selected by the state highway departments and the federal government, and is restricted to the construction, reconstruction, and improvement of highways designated as parts of a federal-aid system of highways. All proposed projects must be approved by the federal government before there can be a commitment of federal-aid funds and, except for certain projects in the federal-aid secondary system, the work for an approved project must also be approved before the state highway department may proceed with the undertaking, if the department is later to claim reimbursement of the federal share of the costs.

For almost three decades, use of federal-aid highway funds was restricted to rural portions of what now constitutes the federal-aid primary system, an extensive network including most of the country's important roads. Prior to the passage of the 1944 Federal-Aid Highway Act, most of the highway construction undertaken as part of a bureau-state cooperative highway program was lo-

cated in rural areas and generally had little or no effect on existing utility installations. The 1944 act authorized: (1) continuance of a federal-aid primary highway system, (2) establishment of a federal-aid secondary highway system, (3) establishment of a national system of interstate highways to be a portion of the primary system, and (4) extensions of these three systems within urban areas. It also modified the definition of *construction* to include "locating, surveying, and mapping, costs of right of way, and elimination of railroad hazards."

The federal-aid highway acts of 1956 and 1958 authorized a tremendously enlarged highway program, which, in its entirety, will be the greatest peacetime construction program in history. While extending at an increased rate the traditional aid for primary, secondary, and urban highway improvements, the 1956 act authorized federal aid over a 13-year period for completion of the interstate system, and established a federal highway trust fund to receive federal highway user taxes from which funds for federal highway aid are disbursed. In 1958, the laws relating to highways were revised, codified and re-enacted as Title 23, US Code, Highways.

Reimbursement for Utility Relocation

Section 123 of Title 23 provides a statutory basis for federal-aid reimbursement to the states of a part of the cost of relocating utility facilities necessitated by highway improvements. The section is quoted in full:

Relocation of Utility Facilities

(a) When a state shall pay for the cost of relocation of utility facilities necessitated by the construction of a project on the federal-aid primary or secondary systems or on the interstate system, in-

cluding extensions thereof within urban areas, federal funds may be used to reimburse the state for such cost in the same proportion as federal funds are expended on the project. Federal funds shall not be used to reimburse the state under this section when the payment to the utility violates the law of the state or violates a legal contract between the utility and the state. Such reimbursement shall be made only after evidence satisfactory to the Secretary [of Commerce] shall have been presented to him substantiating the fact that the state has paid such cost from its own funds with respect to federal-aid highway projects for which federal funds are obligated subsequent to Apr. 16, 1958, for work, including relocation of utility facilities.

(b) The term *utility*, for the purposes of this section, shall include publicly, privately, and cooperatively owned utilities.

(c) The term *cost of relocation*, for the purposes of this section, shall include the entire amount paid by such utility properly attributable to such relocation after deducting therefrom any increase in the value of the new facility and any salvage value derived from the old facility.

Policy and Procedure Memorandum 30-4 of the Bureau of Public Roads, commonly referred to as PPM 30-4, was developed for the administration of the foregoing section. The purpose of PPM 30-4 is to prescribe the extent to which federal funds may be applied to costs incurred by, or on behalf of, utilities in the adjustment of their facilities required by the construction of highway projects.

Essentially, three basic principles are involved within the framework of PPM-304. First, just compensation will be paid where there is a taking of property. This principle is reflected in the provisions of Paragraph 3a(1), which authorizes federal reimbursement where the utility has a right of occupancy in its existing location by

reason of holding the fee, an easement, or other property interest. A utility that is required by necessities of highway construction to give up such a real property right is being deprived of its property, and due process of law requires payment of just compensation. PPM 30-4 accordingly provides a method for federal participation in reimbursement of utility costs incurred under such circumstances.

The second principle is that federal funds may be applied where the state pays for utility relocation and such payment does not violate either the state law or any legal contract between the utility and the state. This principle, which is expressed in Paragraph 3a(2), is a restatement of the requirements of Section 123, Title 23, US Code, Highways. Because the statute establishes these conditions as prerequisites for federal reimbursement, PPM 30-4 requires the states to certify the existence of the required facts.

The third principle is one that is inherent in the administration of any statute—namely, that the administering agency has the authority, and the duty, to carry out its statutory responsibilities in a manner reasonably adapted to accomplish the purposes of the statutes. This is the principle that, on the one hand, authorizes the administering agency to take measures reasonably adapted to carry out the objectives of the legislation and, on the other hand, imposes the responsibility on the agency to protect the government's interest. This is the legal principle that underlies the administrative requirements in PPM 30-4 which are designed to insure that federal funds will be devoted only to the intended and authorized purposes.

Another requirement expressed in PPM 30-4 needs to be explained. Paragraph 1c provides, in substance, that where agreement and payment standards authorized under state law vary from those in PPM 30-4, the more restrictive standards shall govern. Consequently, if the state-utility agreement and payment standards are more liberal than those authorized by PPM 30-4, the federal government must reimburse on the more restrictive basis of the PPM, because it cannot exceed its own authority under Section 123c of Title 23, which defines cost of relocation as including the entire amount paid by the utility properly attributable to the relocation. If, on the other hand, state reimbursement is more restrictive than that authorized by PPM 30-4, then the federal government, in exercising its authority, is limited by the amount actually paid by the state, because Section 123a limits federal reimbursement to the amount actually paid by the state.

Provisions of PPM 30-4

The administrative provisions of PPM 30-4 under the headings "Definitions," "Rights of Way," "Preliminary Engineering," and "Construction" are not different from provisions that apply to federal-aid highway projects not involving utility adjustment. The provisions under "Agreements and Authorizations" are pertinent only to projects involving utility adjustments. The objectives of these provisions are to insure that there is complete understanding between the Bureau of Public Roads and a state highway department, as well as between the state and any affected utility, as to the extent of the work under separate phases, as

to the plans, specifications, and estimates therefor, the manner of construction, and the bases of payments, before the bureau gives approval and authorizes that the work may proceed. With these understandings, approvals, and authorizations, all parties are protected insofar as the use of, and the reimbursement from, federal-aid highway funds is concerned.

The remaining provisions under the headings "Recording of Costs," "Reimbursement Basis," "Labor, Materials, and Supplies," "Equipment," "Transportation," and "Utility Bills" are statements of standard practices of cost accounting that have been established through experience and found satisfactory for both payment and reimbursement purposes. A manner of recording costs that is in accordance with the system of accounts prescribed for the utility company by a state or a federal regulatory body is also proper for federal-aid highway work.

Reimbursement Problems

A primary responsibility assigned the Bureau of Public Roads is to limit the use of federal-aid highway funds to highway construction. With regard to utility adjustments, federal-aid funds cannot be expended for work that increases the capital value of the physical plant or operating facilities of a utility. Nor can federal-aid funds be expended to pay for any of a utility's normal operating costs.

Although the objective of restoring a utility to its former status is not complicated, the details of carrying it out became involved because of complicated situations. Where, for example, the utility relocation involves merely the picking up of property from one location and setting down

the identical physical property in another location, the cost of relocation consists of the labor, engineering, and other incidental necessary expenses, including, in some instances, rights of way. On the other hand, where, as part of the relocation, the utility constructs new, additional, or improved facilities at the new location, it is necessary to determine how much of the expense incurred is properly attributable to the relocation necessitated by the highway improvement. In such instances, it becomes necessary to determine the extended life of the new facility, over that of the old, and the salvage value of the old facility; for if credit for these items is not obtained, the utility's position will be enhanced rather than restored.

The policy adopted by the Bureau of Public Roads on this point is expressed in Paragraph 7i of PPM 30-4. The policy is premised on the concept that the use of new materials does not alone create an increase in the value of a utility facility. Rather, the new materials are viewed as a component of the facility, and increase in value is recognized only where this component can be expected to remain in useful service beyond the time when the overall utility facility, of which it is a part, is replaced.

Advance Planning

Although items of reimbursement may be of great interest, the matters of liaison and advance planning are equally important in any successful undertaking. It is essential for utility companies and state highway departments to make suitable arrangements to begin the planning of any needed utility adjustments well in advance of the associated highway construction.

Such an arrangement will provide ample opportunity and time for utilities to design the needed adjustments to fit the proposed highway design and construction; provide the necessary funds within their budgets; procure the materials, supplies, and equipment; and organize their forces and work schedules.

The Bureau of Public Roads' Policy and Procedure Memorandum 20-11.1, issued on Oct. 10, 1958, discusses the problem of construction planning in relation to right-of-way clearances, utility adjustments, and railroad adjustments. The memorandum is the result of a determined effort to insure adequate advance planning and to make it as simple as possible to proceed with the necessary adjustments well in advance of a highway project construction. The memorandum recognizes the excellent record of utility

companies in performing their work by force account methods through utilization of company forces and equipment, and waives the requirement of proof of public interest in this regard.

Conclusion

The Bureau of Public Roads considers the accommodation, adjustment, relocation, or installation of utilities as being associated with a development program that will bring great benefits to the general public and all its varied interests. The federal-aid highway program is being advanced in the best interests of the general public, with attention to interests of highway users being given first order of responsibility. Careful consideration is also given to the special interests of any segment of the economy to insure that such interests are reasonably treated and left intact.



Turbidimetric Control of Filter Effluent Quality

—John L. Cleasby—

A contribution to the Journal by John L. Cleasby, Asst. Prof. of Civ. Eng., Iowa State Univ., Ames, Iowa.

THE use of high filtration rates increases the need for an accurate, sensitive device to monitor the quality of filter effluent continuously and instantaneously. The filter influent of surface water treatment plants that practice prechlorination and the filter influent of ground water treatment plants are relatively free of bacteria. In these treatment plants, the effluent turbidity is frequently the best index of filter performance; bacterial evaluation is a secondary index. Degradation of effluent turbidity results in a corresponding degradation in bacterial quality in those waters containing a significant bacterial population.

An accurate, sensitive turbidimeter that continuously reads and records data should be of great value to the water supply industry. Such an instrument* has been recently developed. The low-range turbidimeter has been used extensively in plant scale and pilot plant research on high-rate filtration at Iowa State University, Ames, Iowa, and at the Ames treatment plant. It has great potential value in filter control.

Description of Turbidimeter

The turbidimeter (Fig. 1) is a 4-in. ID plastic tube, approximately 4 ft

long. The water enters at the bottom and overflows through a pipe connection approximately 1 ft from the top. At the top of the tube above the overflow are a light source and two lenses. The light source sends a parallel beam, approximately 2 in. in diameter, down through the water column. A number of photoelectric cells are mounted around the circumference at approximately the middle of the tube. The interior of the tube is dull black to prevent the reflection of light from the bottom or sides of the tube. As the light beam passes through the water column, light is scattered in accordance with the Tyndall effect by the turbidity particles. The scattered light received by the photoelectric cells results in the generation of a small electric potential that is measured on a highly sensitive galvanometer. The intensity of the scattered light is in proportion to the number of particles in suspension. A potentiometer in parallel with the galvanometer can be adjusted to cause any desired portion of the output of the photoelectric cell to be shown on the galvanometer. If desired, the output can be continuously recorded on a recording millivoltmeter.

The potentiometer can be adjusted to give a 1-mm galvanometer deflection for a 0.02-unit turbidity. With this potentiometer setting, it is a simple matter to read turbidity to a fine-

* C. R. Low-Range Turbidimeter, made by Hach Chemical Co., Ames, Iowa.

ness of 0.02 unit. It is a well recognized fact, however, that turbidity is a very indefinite quality to measure. Many problems prevent the accurate determination of an absolute turbidity level for any water sample.

Low-Level Turbidity Measurements

The standard method for the determination of turbidity is the Jackson candle method.¹ The Jackson candle turbidimeter, used for the method, is effective only for turbidity concentrations greater than 25 units. Other instruments may be used, but the standard method requires that they be calibrated against the Jackson candle turbidimeter. To do this, a suspension with more than 25 units of turbidity is made up and diluted to the level required for the instrument to be used. This requirement is the reason for inconsistencies in calibration.

First, inconsistencies result from the visual observation required with the Jackson candle turbidimeter in the preparation of initial standards. Second, the standard method suggests the use of the same type of suspended matter as will be observed in subsequent use of the instrument; thus, the nature of the suspension may vary from time to time. Third, there is a possibility that the suspension used in calibration may partially dissolve when it is diluted to less than 1 unit. Fourth, the changes in particle size or other characteristics which may occur during dilution and handling of the standards as a result of natural coagulation may result in inconsistencies.

Furthermore, for accurate measurement of turbidities of less than 1 unit, a water with zero turbidity must normally be used in preparing the desired

dilutions for calibration. An instrument such as the Baylis turbidimeter requires that a dilution water with an absolute zero turbidity be used in preparation of the standards. Any unknown turbidity in the dilution water will result in inconsistencies.

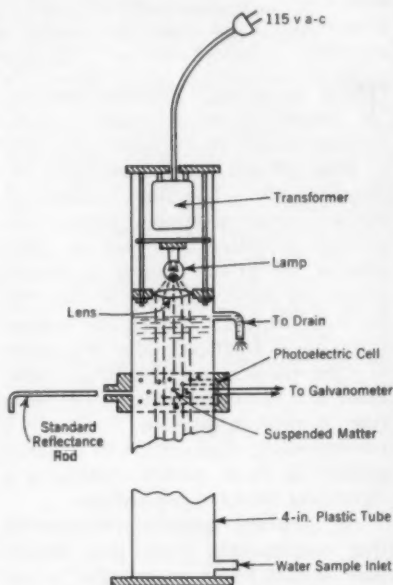


Fig. 1. Low-Range Turbidimeter

Light from the lamp is scattered by turbidity particles and received by photoelectric cells that generate an electric potential, which is measured on a galvanometer.

Advantages of New Turbidimeter

An advantage of the new instrument is that the preparation of the standards for calibration does not depend on water with zero turbidity. The output of the photoelectric cell can be observed for the dilution water, and the increase in output can then be

observed for any desired level of added turbidity.

The photoelectric-cell output of the turbidimeter is essentially zero for water with zero turbidity. Various methods were used in an attempt to prepare a zero-turbidity water, including low-rate sand filtration, recycled filtration of diatomaceous earth, membrane filtration of distilled water, and recycled activated-carbon filtration. The last method produced a water that most nearly had zero turbidity. The filter was a perforated copper pipe septum, wound with felt cloth 0.5 in. thick. Activated carbon was the filtering medium, and water was recycled through the turbidimeter and the filter. The galvanometer soon reached a constant reading, equivalent to approximately 0.05 unit of turbidity. This photoelectric-cell output may result partially from the reflected light reaching the photoelectric cells within the meter.

Various other problems involved in low-level turbidity monitoring are solved with the new turbidimeter. Some photoelectric turbidimeters measure both transmitted light and scattered light. Although this measurement is desirable for observing the optical properties of suspensions with medium to high concentrations of turbidity, the change in transmitted light is insignificant when waters with very low turbidity concentrations are measured. One unit of turbidity reduces transmitted light by about 1 per cent, and it is difficult to measure accurately such a small difference between two relatively large values. In contrast, an increase of 0.1–1.0 unit of turbidity increases the scattered light approximately tenfold. Such a change can be measured accurately. For this reason,

the new meter attempts only to measure scattered light.

The measurement of transmitted light presents some additional problems. Color in water from causes other than turbidity will also reduce the intensity of the transmitted beam and will be measured as turbidity. The manner in which the light beam enters the water may also cause false turbidity readings. In some photoelectric turbidimeters, the light beam passes through a window into the water, and the scattered and transmitted lights pass through other windows to the photoelectric cells. Submerged windows of this type tend to collect air bubbles or to become fouled with slime. This fouling will scatter the light of the incoming beam and will give false high readings of scattered light. Fouling will also reduce the intensity of the transmitted beam. In the new turbidimeter, the light beam enters directly through the surface of the water, which is constantly overflowing and is therefore self-cleansing.

The windows in the new turbidimeter, through which the scattered light passes, may become fouled. This will result in slightly low readings, but not in false high readings. The new meter is furnished with a glass reflectance rod that can be used to check the potentiometer adjustment and proper functioning of the meter. The potentiometer adjustment can be measured by the galvanometer deflection (in millimeters) caused by the light reflected by the reflectance rod. This value is called the sensitivity. If the sensitivity decreases because of fouling, the window can be cleaned, or the potentiometer can be adjusted to increase the sensitivity to the desired level.

Calibration of Turbidimeter

The manufacturer of the low-range turbidimeter anticipated a possible correlation between the light reflected by the reflectance rod and the light reflected by some standard unit of turbidity suspension. This relationship varies for different waters because of the variations in particle characteristics and size distribution. Therefore, the relationship must be observed for the particular suspension being evaluated.

During the pilot plant studies with Ames filter influent water, the relationships of turbidity, sensitivity, and increase in galvanometer reading were carefully studied. Backwash water from the filter was used in preparing standard solutions with turbidities of 1, 0.5, and 0.25 units which were passed through the turbidimeters. The sensitivity of the turbidimeters was carefully observed. The increase in the galvanometer reading for the various turbidities was carefully noted at several different sensitivities.

The galvanometer reading increased in direct proportion to the added turbidity, and the galvanometer reading for a given turbidity increased in direct proportion to the sensitivity. Because both turbidity and sensitivity have a linear effect on the galvanometer reading, the calibration for a particular water may be stated in terms of millimeters of galvanometer deflection per unit turbidity per millimeter of sensitivity. For the Ames filter influent water, this value was found to be 2.95.

Plant Control

During the course of the current research, several important uses of the new turbidimeter have become apparent, three of which will be discussed here:

1. *Selection of proper time to backwash filter.* Many plant operators and regulatory personnel are lulled into a false sense of security by the use of a standard rate of filtration of 2 gpm/sq ft. Under some conditions, water with excessive turbidity may pass the filter at low head losses. The use of a continuous monitoring device such as the new turbidimeter will permit ready observation of the proper time to backwash a filter to maintain the desired water quality.

2. *Study of feasibility of high filtration rates.* Because a continuous measurement of the quality of filter effluent water may be obtained with the new meter, comparative studies at standard and high filtration rates can be made. Such a study at Ames has resulted in the adoption of rates as high as 4 gpm/sq ft, as the demand necessitates. Moreover, the need for plant expansion has been delayed a number of years.

3. *Observations of effect of plant operational practices on effluent-water quality.* Observations during the research have indicated that any operational practice that results in a rate increase on a partially dirty filter will cause considerable amounts of previously deposited material to be flushed through the filter. The filter recovers after a period of time, but such periods of high turbidity would most likely go undetected with only periodic sampling. Automatic rate control that allows the filtration rate to increase and decrease as the level of the clear well fluctuates is therefore subject to question. It may result in reduced water quality, particularly where runs are fairly long, and significant rate increases may occur when a dirty filter is used.

Conclusions

1. The adoption of higher filtration rates increases the need for continuous monitoring of filter effluent.

2. Turbidity measurement is the best means of continuous control of filter effluent quality.

3. The new, low-range turbidimeter, which provides an accurate, con-

tinuous record of turbidity, is a welcome addition to plant control instrumentation.

Reference

1. *Standard Methods for the Examination of Water and Wastewater*. APHA, AWWA & WPCF. New York (11th ed., 1960).

Coagulant Aids for Potable-Water Treatment

The USPHS Technical Advisory Committee on Coagulant Aids for Water has added the following product to the list of those that may be used for water treatment—in the concentration recommended by the manufacturer—with no adverse physiological effects on water consumers: Aquafloc 422.

The committee's findings bear only on the health aspects of the use of such products and do not constitute an endorsement or indicate their effectiveness in the proposed use. Coagulant aids on the committee's list as of Sep. 14, 1960, are:

Manufacturer	Product	Max. Concentration Recommended by Manufacturer ppm	Manufacturer	Product	Max. Concentration Recommended by Manufacturer ppm
Allyn Chemical Co.	Claron Claron No. 207	1.5 2	Hagen Chemicals & Controls, Inc.	Coagulant Aid No. 11 Coagulant Aid No. 18 Coagulant Aid No. 801 Coagulant Aid No. 952	4 15 6 8
North American Mogul Products Co.	Mogul CO-980 Mogul CO-982 Mogul CO-983	2 1.5 1	Hercules Powder Co.	Carboxymethyl-cellulose	1
The Burtonite Co.	Burtonite No. 78	5	Illinois Water Treatment Co.	Illico IFA 313	10
Dearborn Chemical Co.	Aquafloc 422	1	Kelco Co.	Kelgin W Kelcosol	2 2
Dow Chemical Co.	Separan NP10, potable-water grade Purifloc N17	1 1	National Aluminate Co.	Nalcolyte No. 110	5
E. F. Drew & Co., Inc.	Drewfloc	6	The Permutit Co.	Permutit No. 65 Permutit No. 66 Permutit No. 67 Permutit Wisprofloc-20†	2 2 4 5
Electric Chemical Co.	Ecco Suspension Catalyzer No. 146	3.5	Stein, Hall & Co.	Jaguar	0.5
Hagan Chemicals & Controls, Inc.	Coagulant Aid No. 2 Coagulant Aid No. 7	1 0.75			

* 1.8 ppm alum, 0.5:10 lime.

† Replaces Permutit No. 68.

Water Disinfection Practices in the United States

—Edmund J. Laubusch—

A contribution to the Journal by Edmund J. Laubusch, Tech. Mgr., Chlorine Institute, New York, N.Y.

IN 1908, chlorinated lime was first used in the United States as a water disinfectant on a plant scale for the Boonton reservoir supply of Jersey City, N.J. The then unorthodox procedure was exhaustively reviewed in action before the New Jersey courts, whereupon it was demonstrated to have no deleterious effects and was accepted as a public health safeguard. In 1909, chlorinated lime was used in the Poughkeepsie (N.Y.) water supply, and by 1914, supplies totaling at least 800 mgd were being similarly chlorinated.¹

Also in 1909, liquid (compressed gas) chlorine in steel containers was introduced as an article of commerce, but it was not employed on a plant scale until 1912, when it was used at the Niagara Falls (N.Y.) filter plant. Added impetus to the popularity of liquid chlorine for water treatment came about through the proprietary availability of chlorine gas-dispensing equipment in 1912-13. The early commercial availability of liquid chlorine and suitable dispensing equipment is largely responsible for the rapid adoption of chlorine disinfection of water as a public health measure. The inherent weakness of chlorinated lime as a water disinfectant, and consequent uncertainties and inconsistencies in its effects, were not overcome until 1928, when stable, high-test hypochlorites

containing 70 per cent available chlorine were provided. Finally, the introduction and general acceptance of orthotolidine as a color indicator of residual chlorine concentration during the period 1909-18 established chlorine disinfection as a scientific, routine plant procedure.

During the past half century of experiment, refinement, and understanding of water chlorination processes, certain policies and practices have been developed to insure the effectiveness of chlorine for disinfection and numerous other applications in water treatment technology. Today, no other water treatment process is so popular and so widely used in the United States. Although not all of the public health protection afforded by current water treatment technology can be attributed to chlorine, its unquestionable value is evidenced by its increasing use (Table 1). In 1948, the latest year for which USPHS nationwide statistics (unpublished) are presently available, more than 50 per cent of all existing water treatment facilities employed chlorination as the only treatment. Moreover, about 88 per cent (6,137) of the existing facilities, serving more than 80,000,000 people, employed chlorine disinfection alone or in conjunction with other water treatment processes. This represented more than 96.3 per cent of the population served by all

community water systems, both with and without treatment, and 96.9 per cent of the population served with treated water.³ The application and benefits of chlorine in water treatment for purposes other than disinfection are similarly well established, although disinfection constitutes the most important use of chlorine.

Standards of Quality

At present, no standards of water quality are used internationally. It is interesting to note, however, that in

of, the American water supply industry—has refrained from promulgating or enforcing official standards of water quality. But it did, in 1946, adopt the USPHS Drinking Water Standards as its *recommended* standard of water quality for all public water supplies. Notwithstanding, except to the extent that the standards can be enforced by the federal government or by state governments that may have accepted them, adoption and acceptance by AWWA members is on a voluntary basis.

Another effective stimulant to the development of improved water quality and standards of treatment was the evolution of standard methods of water analysis, the need for which was recognized by various leaders in the water supply field and by members of APHA as early as 1894. As a result, in 1905, the first edition of *Standard Methods of Water Analysis* was published, which has evolved into the eleventh edition of *Standard Methods for the Examination of Water and Wastewater*.⁵ These standards include analytic procedures for chlorine demand and residual determinations, and are known and respected throughout the world. They, too, however, are not applied internationally.

The United States government has no control over the quality of public drinking water, except for the water used on interstate carriers. Authorization for this control and jurisdiction by USPHS are provided by the federal Quarantine Regulations Act. By this authority, USPHS Drinking Water Standards were published in 1914, 1925, 1942, and 1946,⁶ and are legally binding only in regard to water used on common carriers subject to the federal quarantine regulations and engaged in interstate commerce. The

TABLE 1
Increase in Use of Water Chlorination in
the United States*

Year	Water Treatment Facilities		Population Served —1,000,000's		References
	Total No.	No. Using Chlorination	Total	With Chlorinated Water	
1910	557 +	18 +	13.91 +	1.18 +	1, 2
1915	830	340	22.74	15.58	2
1930	3,015†	2,917†			2, 3
1940	5,372	4,650	74.31	63.50‡	3, 4
1948	6,986	6,137	83.25	80.63	3

* Data are not strictly comparable because of slight variations in statistical reporting and analytic procedures.

† Data appear to be questionable because of the unlikely high percentage of available treatment facilities employing chlorination.

‡ Estimated value.

1955 the World Health Organization moved to develop international standards of drinking water quality and approved methods for the examination of water. This effort is still in progress.

One of the most effective stimulants to the development of improved water quality and treatment practices in the United States was the formation, in 1881, of the American Water Works Association. For nearly 80 years, AWWA—indisputedly recognized for its leadership in, and as spokesman

regulations contain no specifications for methods of water disinfection; they refer only to the bacteriologic quality of drinking water, regardless of how the water is treated. USPHS, in cooperation with various state sanitary engineers and others concerned, has published a corollary report⁷ designed to assist those who, by requirement or voluntary action, provide water complying with the USPHS standards. The report contains numerous references and recommendations pertaining to all aspects of water disinfection treatment and control processes, some of which will be discussed here.

Control of Quality

Control of public water quality and, accordingly, of treatment methods is under the jurisdiction of the various states and their political subdivisions. As will be noted later, the degree to which authority is exercised among the states varies. Where interstate waters are involved, some states have organized cooperative agencies or compacts. Some of these groups, usually without enforcement powers, define standards of water quality, often through stream classification. These, undoubtedly, influence the policies and practices of authorities in those states involved but exert no apparent influence on nationwide water treatment practices or objectives.

One of the most prominent guides to water utility practice was prepared in 1950-53 by a committee of the Great Lakes-Upper Mississippi River Board of State Sanitary Engineers, including health department representatives from New York, Pennsylvania, Ohio, Michigan, Indiana, Illinois, Wisconsin, Minnesota, Iowa, and Missouri. The report of this committee,⁸ the "Ten-State Standards," is in-

tended "to serve as a *guide* in the design and preparation of plans and specifications for public water supply systems, to suggest limiting values for items upon which an evaluation of such plans and specifications may be made by the reviewing authority, and to establish, as far as practicable, uniformity of practice among the several states." Statutory requirements and legal authority pertaining to public water supplies are not uniform among the signatory states. Accordingly, there naturally continue to be variations in the extent to which the Ten-

TABLE 2

*States With Published Recommendations or Minimum Standards for Public Supply Systems**

Alabama	Indiana†	North Carolina
Arkansas	Kansas	Oregon
California	Kentucky	Pennsylvania†
Colorado	Louisiana	South Carolina
Connecticut	Minnesota†	Texas
Delaware†	Missouri†	Utah†
Florida	Nebraska	Washington
Georgia	New Jersey	West Virginia†
Hawaii	New Mexico	Wisconsin†
Illinois†	New York†	Wyoming

* Standards are currently being prepared by Maine, New Hampshire, and Oklahoma authorities. The status of standards in Alaska and Mississippi is unknown.

† States relying largely or exclusively on Ten-State Standards. Iowa, Michigan, Montana, Ohio, South Dakota, and Oklahoma also rely on these standards.

State Standards are applied. In some instances, these standards represent the only published state record on water practice; in others, they have been supplemented or adopted only in part; and in still other instances, state-published recommendations or regulations are not offered. In all states, however, the standards serve as a guide to the reviewing authority in judging the merits of existing or proposed facilities. The impact of the Ten-State Standards on greater uniformity of water utility practices in

the United States is an essential matter of record.

State Authority

Probably without exception, all 50 states have legislative authority to exercise control over the sanitary quality of public water supplies and the design and operation of new systems. In most instances, this authority provides that minimum standards of design, construction, operation, and maintenance can be established. At least 60 per cent of the states publish minimum standards. The remainder—including three of the signatory states to the Ten-State Standards—apparently prefer the flexibility of handling each system separately. Of the 30 states that have published standards, 10 have adopted or otherwise use as a guide the Ten-State Standards. Also, six of the fifteen or more states that do not publish minimum standards rely almost exclusively on the Ten-State Standards. Thus, as shown in Table 2, the Ten-State Standards are the accepted review criteria in at least sixteen states. Among the remaining states, no uniform review criteria are apparent.

Raw-Water Quality

Very few of the states appear to have established rigid limiting bacteriologic criteria for an acceptable raw-water source (Table 3). This is true also of the Ten-State Standards, which specify that water sources that can be rendered chemically and bacteriologically safe by reasonable treatment are considered suitable for development. Criteria of safety applied are those specified by the USPHS Drinking Water Standards. Design engineers are thus allowed some de-

gree of flexibility in exercising professional judgment in the selection of sources for public water supply development.

On the other hand, as the data illustrate, there is fairly good current agreement as to limiting concentrations of coliform organisms in water sources that can be rendered bacteriologically safe by conventional water treatment processes. A MPN of 20,000 for coliform organisms appears to be accepted as the maximum safe limit for pre-chlorination, presedimentation, rapid sand filtration (or the equivalent), and continuous postchlorination. But data reported by Walton⁹ indicate that:

Coagulation, sedimentation, and filtration, as currently practiced, are inadequate for treating waters having densities of coliform organisms in excess of approximately 50 per 100 ml. Plant superintendents must provide supplementary treatment by continuous and adequate chlorination for any plant treating waters containing substantial densities of coliform organisms.

Among the factors included by Walton to be considered in evaluating a plant's capacity to treat waters with high concentrations of coliform organisms are: locations at which chlorine is applied, residual chlorine levels maintained in the plant effluent, frequency of chlorine residual tests, and the availability of adequate standby chlorinating equipment. These and other factors will be discussed.

Disinfection Policies

As already mentioned, USPHS water quality standards do not specify minimum treatment, only limiting criteria of chemical and bacteriologic safety. The corollary *recommendations* of USPHS referred to previ-

ously⁷ are based on the MPN's of coliform organisms in raw water outlined in Table 3. In general, it is recommended that *all* surface supplies intended for drinking and culinary purposes be chlorinated as a minimum safeguarding treatment.

The Ten-State Standards require that all surface water supplies and all ground water supplies of unsatisfactory sanitary quality be disinfected. Where the disinfectant is a chemical or agent other than chlorine or various chlorine-containing compounds—

TABLE 3
Limiting Bacteriologic Criteria for Development of Surface Supply Sources

Authority	Local Classification of Water	Unit	Limiting Concn. of Coliform Organisms—MPN
Arkansas		avg monthly MPN max. monthly MPN max. monthly MPN	5,000 20% samples > 5,000 5% samples > 20,000
Kansas		max. MPN* max. MPN max. MPN max. MPN	50† 5,000‡§ 20,000‡ > 20,000#
Minnesota	I	avg MPN	50 20% samples > 50*
	II	avg MPN	5,000
	II	max. MPN	20% samples > 4,000**§
	II	max. MPN	100% samples > 2,000
	III	max. MPN	20% samples > 5,000‡ not > 20,000
New York Water Pollution Control Bd.	A1 & A2 A4	avg monthly MPN avg monthly MPN	50† 5,000 20% samples > 5,000**
USPHS Recommendations	Group II Group III Group III Group IV Group IV Group IV Group V	avg monthly MPN avg monthly MPN max. monthly MPN avg monthly MPN max. monthly MPN max. monthly MPN avg monthly MPN	50† 5,000** 50% samples > 5,000 5,000‡§ 20% samples > 5,000 5% samples > 20,000 > 20,000#
TVA	I II III	geometric avg MPN geometric avg MPN geometric avg MPN	50† 5,000** 20,000‡

* For ground water.

† Minimum treatment is chlorination.

‡ Auxiliary treatment plus postchlorination required.

§ Prechlorination recommended.

|| Prechlorination required.

Probable undesirable source.

** Minimum treatment is filtration plus postchlorination.

liquid (gas) chlorine, hypochlorite, chlorine dioxide, or chlorine and ammonia—specific authorization by the state reviewing authority also is required. Again, satisfactory disinfection is judged on the basis of conformance with USPHS bacteriologic requirements for interstate water supplies.

Obviously, as the data in Table 1 indicate, all states do not require chlorine disinfection of all public surface

Chlorine Application

Because the location of the points at which chlorine is applied depends on several factors—characteristics and chlorine demand of the water being treated, specific objectives of treatment, contact time, and effects of chlorine addition on other treatment chemicals and processes—no uniform rules can be applied to all treatment installations. Generally, the points at which

TABLE 4
State Practice Regarding Postchlorination of Surface Supplies

Recommended or Required (Specified)	Recommended or Required (Implied*)	Not Specified or Implied†	Status Unknown
Alabama	Delaware	Arkansas	Alaska
Florida	Illinois	California	Arizona‡
Louisiana	Indiana	Connecticut	Maine‡
Massachusetts‡	Iowa‡	Colorado	Maryland‡
New Jersey	Kansas	Georgia	Mississippi
New Mexico	Michigan‡	Hawaii	New Hampshire‡
North Carolina	Minnesota	Idaho‡	Rhode Island‡
North Dakota‡	Missouri	Kentucky	Virginia‡
South Carolina	Montana‡	Nebraska	
Tennessee‡	New York	Nevada‡	
Texas	Ohio‡	Oregon	
Washington	Oklahoma‡	Vermont‡	
Wisconsin	Pennsylvania		
Wyoming	South Dakota‡		
	Utah		
	West Virginia		

* As supporter of Ten-State Standards.

† Presumably, postchlorination is used when needed to comply with USPHS Drinking Water Standards.

‡ No published standards.

and ground water supplies. The practices of various states with respect to surface supplies are shown in Table 4. The scope of data analysis by the author did not extend to ground supplies; undoubtedly, state policies in this regard are more flexible. At least 60 per cent of the states require or recommend terminal disinfection of surface water sources as a minimum safeguarding treatment, regardless of raw-water quality.

chlorine is applied should be such that thorough mixing and sufficient contact time is obtained. USPHS recommendations specify that chlorine be applied continuously to the filter effluent; the Ten-State Standards specify that the plant design include provision for chlorine application to the raw water, applied water, and the filtered water or high-lift pump suction.

It is abundantly evident that practices vary considerably. In the early

days of chlorine disinfection, terminal treatment was the customary procedure. Today, the application of chlorine in various stages of water treatment ("split chlorination"), and even in the potable-water distribution system, is increasingly employed for disinfection and other purposes. As one would expect, few states have a fixed policy in this regard. Authorities usually relate requirements to local conditions and treatment objectives.

Kansas when the MPN of coliform organisms in raw water exceeds 5,000, and it is required when the MPN exceeds 20,000. Chlorine practices in Minnesota are related to "factors of safety" as a function of the MPN of coliform organisms, with prechlorination usually specified when the average MPN exceeds 4,000 or where it varies up to approximately 100 per cent of an average MPN of 2,000. For waters of inferior quality, double chlo-

TABLE 5

Recommendations or Requirements for Minimum Postchlorination of Surface Supplies

Authority	Treatment Specified
USPHS	0.2-ppm free residual after 20-min contact, or 2.0-ppm combined residual after 3-hr contact; 0.05-0.10-ppm free residual or 1.0-2.0-ppm combined residual maintained
Ten-State Standards	as necessary for bacteriologic safety
Connecticut	0.2-ppm free residual after 10-min contact; 0.5-ppm combined max. residual
Florida	residual* maintained
Iowa	30-min contact
Kansas	30-min contact (max. MPN 20,000)
Kentucky	30-min contact
Massachusetts	0.2-ppm free residual
Minnesota	0.4-ppm residual† after 3 hr maintained
Nebraska	0.05-ppm free residual maintained
Nevada	1-ppm residual† after 20-min contact
New Mexico	0.2-0.4-ppm residual† maintained
Texas	0.20-ppm free residual after 20-min contact or >0.20-ppm combined residual after 3-hr contact maintained
Washington	0.2-0.3-ppm residual† maintained
Wyoming	0.1-ppm residual† maintained

* Amount and type not specified.

† Type not specified.

Although the benefits of prechlorination are similarly well established, only a few states have an overall policy in this regard. In New Jersey, for example, prechlorination is required at all rapid sand filter installations, but not at slow sand filter installations. In Kansas and Minnesota, prechlorination policies are based on density of coliform organisms in the raw water. Prechlorination (minimum 30-min contact at pH > 7.5) is recommended in

rination, superchlorination, or other degrees of chlorination also may be specified to yield a finished water having a zero MPN.

Contact Period

The efficacy of chlorine disinfection depends on the attainment of a specified chlorine residual after a specified contact period. The character and magnitude of the residual and the contact period are largely reciprocally in-

terdependent, and minimum conditions for obtaining the residual vary among different biologic species. Based on research activities, USPHS recommendations as to contact period provide for a minimum of 20 min where free residual chlorination is practiced, and a minimum of 3 hr where combined residual chlorination is practiced.

Contact periods are not specified in the Ten-State Standards, except that a minimum of 30 min is specified where free residual chlorination is the sole treatment. Instead, a maximum possible contact period is recommended which is consistent with the operational effects of pH, ammonia content, taste-producing substances, temperature, and other pertinent factors. It is of interest to note that, in keeping with recent trends, chlorine-ammonia treatment is not generally recommended except by approval of the reviewing authority. Only a few states actually specify minimum chlorine contact periods, as shown in Table 5.

Residual Chlorine

As are policies on chlorine contact periods, policies on residual chlorine requirements are of necessity flexible and quite varied. The usual requirement is that conditions be such that a fixed, minimum residual is obtained which, on the basis of local experience, can be demonstrated to yield a water of satisfactory bacteriologic quality.

USPHS data⁷ indicate that minimum free and combined residuals of 0.2 ppm and 2.0 ppm should exist after 20 min and 3 hr contact, respectively. According to the data, a minimum free residual of 0.05–0.10 ppm or a combined residual of 1.0–2.0 ppm should generally be maintained at distant points in the distribution system.

Moreover, during times of existing or threatened waterborne disease outbreak, and during abnormal or emergency periods of operation, it is recommended that a minimum free residual of 0.1 ppm or a combined residual of 2.0 ppm be maintained in *all* parts of the distribution system. Texas authorities specify a 1.0-ppm *free* residual, where possible, under such conditions. A discussion of the value of chlorine residuals for treatment of potable-water systems may be found in the February 1959 issue of the JOURNAL.¹⁰

Only a few states specify minimum requirements for contact periods and chlorine residuals, as shown in Table 5. In many instances, the residuals pertain to the potable-water distribution system.

Effluent Quality

Regardless of recommendations or minimum requirements pertaining to chlorine residuals, the ultimate proof of satisfactory treatment sought by all authorities is that the water served to consumers is bacteriologically safe. The standards usually referred to by state authorities are those of USPHS. In a few instances, these standards are slightly modified or supplemented as deemed appropriate by the approving authority.

Test Frequency

Most state authorities do not specify how frequently tests for residual chlorine should be made. USPHS recommendations call for tests of the finished water at the treatment plant at least once every 8 hr; at regular sampling points in the distribution system, at least three times weekly. Texas authorities follow this practice. Minnesota authorities establish test fre-

quency as a function of raw-water quality: for MPN's of 0-300, tests are at 12-hr intervals; for MPN's up to 5,000, at 2-hr intervals; and for MPN's up to 20,000, at 1-hr intervals. Connecticut and Nebraska authorities require that tests be made at least daily at representative points throughout the

feed chlorinating equipment be such that a minimum chlorine residual of 2 ppm be attainable after a 30-min contact period, even when maximum flow rates coincide with anticipated maximum chlorine demands. They also provide that automatic proportioning gas chlorinators be used where the rate

TABLE 6
State Practices on Repaired and New Water Main Disinfection

Recommended or Required (Specified)	Recommended or Required (Implied*)	Not Specified or Implied	Status Unknown
Alabama†	Illinois‡	Colorado	Alaska
Arkansas	Indiana‡	Georgia	Arizona§
California‡	Iowa‡§	New Mexico	Maine
Connecticut	Michigan‡§	Wisconsin	Maryland§
Delaware‡	Minnesota‡		Massachusetts§
Florida	Missouri‡		Mississippi
Hawaii	Montana‡§		New Hampshire
Idaho‡§	Ohio‡§		Rhode Island§
Kansas	Oklahoma‡		Tennessee
Kentucky#	Pennsylvania‡		Vermont
Louisiana	South Dakota‡		Virginia
Nebraska	Utah‡		
Nevada**§			
New Jersey‡			
New York			
North Carolina‡			
North Dakota§			
Oregon‡			
South Carolina			
Texas††			
Washington			
West Virginia‡			
Wyoming‡‡			

* As supporter of Ten-State Standards.

† \geq 50-ppm residual for 24-hr minimum contact.

‡ AWWA recommendations: \geq 10-ppm residual after 24-hr contact, or 50-100-ppm residual for shorter contact periods (employed in at least seventeen states).

§ No published standards.

|| \geq 100-ppm residual for 24-hr contact; \geq 25-ppm residual after 24-hr contact.

\geq 50-ppm residual for 24-hr contact; \geq 25-ppm residual after 24-hr contact.

** \geq 5-ppm residual after 3-hr contact.

†† 40-60-ppm minimum residual after 6-hr contact.

‡‡ 40-50-ppm residual for 3-hr minimum contact; \geq 5-ppm residual after 3-hr contact.

distribution system. Apparently most other state authorities prefer the flexibility of specifying minimum requirements for each situation.

Equipment

Capacity. The Ten-State Standards provide that the capacity of solution

of flow is not reasonably constant or is not manually controlled. USPHS recommendations provide that equipment be of such capacity as to exceed the highest expected chlorine dosage, without the use of standby equipment. They also provide that automatic proportioning gas chlorinators be used at

all larger plants where the rate of flow varies more than 50 per cent more or less than average flow, and where the quality of water is subject to change without warning. Texas authorities follow this practice.

Few states define minimum equipment capacity in their printed guides. Kansas authorities specify capacity equivalent to 20 ppm at maximum flow where prechlorination is indicated, and a minimum of 3 ppm at maximum flow for postchlorination. Nebraska authorities specify equipment capable of maintaining the required chlorine residual (0.05-ppm free residual in the distribution system). Tennessee authorities call for equipment capacity capable of achieving free residual chlorination. Texas authorities require the equipment to provide at least 50 per cent more than the highest anticipated chlorine dosage required. It is assumed that the sixteen states that have adopted the Ten-State Standards follow these recommendations.

Standby units. Both the Ten-State Standards and USPHS recommendations stress the necessity of providing adequate standby chlorinating equipment to insure uninterrupted operation, and stress that spare parts and tools be maintained for emergency replacements or repairs. The Ten-State Standards, however, do not require standby units where chlorination is employed only as a safeguarding treatment. Aside from the states following the Ten-State Standards, suitable standby equipment is specified by at least nine additional states: Alabama, Kansas, Minnesota, New Hampshire, New Jersey, North Carolina, Tennessee, Texas, and Wisconsin.

Disinfection of New Mains

At least 35 states specify that all repaired or new sections of potable-

water distribution systems be suitably disinfected before being put into service. The proof of a water's suitability is its freedom from coliform organisms, but at least seventeen states use the AWWA recommended procedure¹¹ as a working guide. A few states have different standards, as shown in Table 6.

Summary

Chlorine disinfection has been firmly established in the United States over the past 50 years, and there is a trend toward its adoption as a minimum safeguarding treatment for all public surface water supplies.

Except for water used on interstate carriers, authority for control of public water supplies is vested in the various states and their political subdivisions. The degree to which authority is exercised varies considerably among the states, but USPHS standards of quality almost invariably serve as minimum indexes of bacteriologic safety. At least 30 states publish minimum standards for water treatment plant design, operation, and maintenance. Ten of these, and six additional states that do not publish them, have adopted the Ten-State Standards, in whole or in part, as the basis of their minimum recommendations or requirements. Among the remaining states, no uniform criteria are apparent.

Not all states require disinfection of all public surface and ground water supplies, but at least 30 states require or recommend chlorine disinfection of surface waters regardless of raw-water quality. Additionally, at least twelve states specify chlorination as needed to comply with bacteriologic quality standards of USPHS.

Only a few states have fixed policies with regard to points of chlorine appli-

cation, minimum contact periods, and residuals obtained. Apparently, health authorities usually relate these to local conditions and treatment objectives. Almost invariably, the critical test of suitable water disinfection is the demonstration of freedom from coliform organisms according to prescribed test procedures. No state authorities define water quality in terms of residual chlorine content, and only a few apparently have rigid policies on the frequency of tests for residual chlorine.

Only a few states define minimum capacity of chlorinating equipment, but 80 per cent of the states known to publish minimum standards or otherwise to have adopted the Ten-State Standards provide for installation of adequate standby chlorinating equipment to insure continuity of operation during emergencies.

Current state policies, regulations, or recommendations may differ from those reported here. Interested persons should consult with appropriate authorities to ascertain local requirements.

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Artificial Recharge of a Well at Walla Walla

—Robert H. Russell—

A paper presented on May 6 1960, at the Pacific Northwest Section Meeting, Portland, Ore., by Robert H. Russell, Asst. Supervisor, Div. of Water Resources, State Dept. of Conservation, Olympia, Wash.

FOR many years there has been a progressive water level decline in some wells taking water from artesian zones of the Columbia River basalt of the Walla Walla Basin. Because of this decline, it became apparent that the Washington Division of Water Resources, through administrative authority, would eventually have to limit the total annual pumpage of water from the basalt aquifers or implement a program whereby the safe sustained yield could be materially increased. In 1951, a well-spacing program was adopted, designed to maintain a reasonable distance between wells pumping from the artesian zones. This, as well as permitting a greater seasonal lowering of the water table, made possible an increased annual withdrawal from the basin with reasonable assurance that water withdrawn would be replaced annually by natural recharge. Hydrographs of observation wells suggest that the program was generally successful, with the exception of the subbasin in which Walla Walla city Wells 1, 2, and 3 are located, where water levels continued to decline.

This succession of events led the Division of Water Resources to request the cooperation of the USGS

Ground Water Branch in a program to evaluate the possibility of supplementing the water supply in basalt aquifers by artificial recharge through injection wells.

Fortunately, Walla Walla officials approved the use of their water supply facilities for the recharge experiment; otherwise the project would not have been economically feasible, as it would have required an expenditure of about \$100,000 to duplicate the facilities.

Geographic Location

Walla Walla is in the southeastern part of the state of Washington, about 5 mi north of the boundary between Washington and Oregon and in the drainage basin of Mill Creek, a westward flowing tributary of the Walla Walla River. The well in which the water was injected, Well 3, is 5 mi east-northeast of the city, near Mill Creek.

Geologic and Hydrologic Features

The Walla Walla Basin is an east-west-trending topographic and structural basin and lies astride the Washington-Oregon boundary. It is 1,300 sq mi in area and is bounded on the north by the "Touchet slope," on the east by the Blue Mountain up-

lift, on the south by the Horse Heaven uplift, and on the west by a rimrock ledge in the lower valley of the Walla Walla River. All these features are beyond the extent of the easternmost portion of the valley.

The Walla Walla Basin is underlain by a series of lava flows which are a part of the basaltic materials designated the Columbia River basalt. The Columbia River basalt extends in general from the Okanogan highlands south to the east-west-trending mountain systems of central Oregon, and from the Cascade Mountains eastward to the Rocky Mountains in Idaho. The Walla Walla basin lies southeast of the geographic center of that vast basaltic area. In this basin, the sequence of lava flows is of unknown thickness. At Walla Walla Well 3, used as the injection well in the recharge experiment, the basalt is at least 1,169 ft thick. In Mill Creek canyon, east of Well 3, a thickness of about 2,000 ft of basalt is exposed. The rock on which the basalt rests is not exposed, nor has it been reached in drilling in the Walla Walla basin. About 35 mi northeast of Walla Walla, in the Tucannon River Canyon, a few small "windows" of Paleozoic and Mesozoic sedimentary and igneous rocks are exposed beneath the basalt, which in that vicinity is known to be at least 3,000 ft thick. About 50 mi southwest of Walla Walla, in the Umatilla River Basin, the basalt lies upon sedimentary Eocene rock.

East of the injection well the Columbia River basalt is arched to form the Blue Mountains. The west side of the Blue Mountains is essentially a monocline dipping 3-5 deg toward the Walla Walla Basin. The Blue Mountains slope is dissected by steep-sided, narrow canyons as much as

1,500 ft deep. Mill Creek Canyon is one of the deepest of these. The gradient of the streams in general is less than the slope of the basalt beds; therefore, the outcrop pattern of each individual lava flow, as viewed from above, is V-shaped, pointing downstream. This outcrop geometry forms favorable conditions for natural recharge of ground water into the permeable layers cropping out in the stream channels.

In the area adjacent to the recharge well, however, hydraulic boundaries seem to occur in a pattern such that insufficient natural recharge water is reaching that area. As a result of this isolation from natural recharge, the replenishment is less than the withdrawal, as shown by long-term decline in ground water levels in Wells 1, 2, and 3.

Water Occurrence and Movement

From areas of recharge where the basalt is at or near the surface, such as the Blue Mountains, rainwater and snowmelt percolate downward into the porous zones between the tabular flows of basalt. The water in these porous zones moves by gravity down the dip of the basalt toward the center of Walla Walla Basin, which is a few miles southwest of Walla Walla. The basalt aquifers beneath the valley floor locally contain water under sufficient hydraulic head that when tapped by wells it flows at the ground surface or stands but a few feet below the surface.

In the eastern part of Walla Walla Basin, ground water movement is, in general, from east to west—that is, from high to lower parts of the basin. In the area immediately adjacent to Well 3, however, the direction of water

movement has not been determined accurately. Wells for which reliable water level data could be obtained are insufficient to warrant construction of a meaningful water level contour map.

Virtually simultaneous determinations of static water levels were made in Wells 1 and 3 on May 27 and 28, Oct. 1, and Dec. 11, 1957, and on Jan. 11, 1958. The levels at the two wells were about equal. Hence, so far as these measurements showed, no hydraulic gradient existed between Well 3 and Well 1 on those dates. Earlier measurements had shown that for at least a part of the period of record a slight westward gradient existed; however, all those water levels were measured by means of air gages, the sensitivity of which was too low to indicate small differences in altitude. Because these water level comparisons were made on only two wells, they represent conditions only along a straight line connecting Well 3 with Well 1. Obviously the two points are insufficient for determining whether a local gradient exists. It is possible that either a northerly or a southerly component of the true gradient may exist here.

The existence of impermeable boundaries adjacent to the recharge area may have an appreciable, but unevaluated, effect on the direction of water movement. Elsewhere in the Walla Walla area hydraulic boundaries exist.¹

Water Level Fluctuations

There is scant information on the long-term fluctuations of water level in this area. Records of water level measurements show that during the period April 1942 to May 1947 the level in Well 3 declined about 38 ft.

There is relatively large seasonal fluctuation of water level in wells in

and adjacent to Wells 1, 2, and 3. For example, from May to October 1947, the level in Well 1 declined 16 ft and that in Well 3 declined 17 ft. From August 1957 to January 1958, the level in the Roy Frazier well recovered 25 ft, from 74 to 49 ft below the measuring point. A small part of this recovery, however, may have been caused by the recharge operations. Although this well is a little more than 2 mi northwest of Well 3, the short-term hydrograph for the Frazier well suggests strongly that the two are hydraulically interconnected.

The long-term decline in levels points out the need for conservation measures; the decline indicates that more water is pumped from the basalt aquifers in the immediate area than is replaced by natural recharge or movement of water from adjoining areas.

Walla Walla Supply

The public water supply system of the city of Walla Walla is served from both surface and ground water sources. The city has an established right to withdraw 14 mgd from Mill Creek. During a part of the year the city's demand is greater than this maximum withdrawal rate; furthermore, for short periods when Mill Creek receives storm runoff, the water in that stream is too turbid for public supply without treatment by costly filtration facilities. During such periods, the city's demand is satisfied by pumping ground water to supplement the Mill Creek supply.

Surface Water Source

Mill Creek, a perennial stream, rises in the Blue Mountains and flows north and then west to join the Walla Walla River about 6 mi west of Walla Walla. The water supplying the city is withdrawn from Mill Creek at a diver-

sion dam and is then led into a settling basin about 3,000 ft north of the Oregon-Washington boundary and about 11 mi southeast of Walla Walla. This settling basin removes only a moderate amount of the sediment present in the water and contains screens which stop larger debris such as pine needles. The settling basin also enables the operator to maintain a visual check on the turbidity of the water. When the water is too turbid to use, it can be bypassed. When it is being used by the city, it is chlorinated as it leaves the settling basin. From the settling basin the water flows

reservoir, the pressure in the 20-in. line is about 18 psi.

Ground Water Source

Four wells, ranging in depth from 789 to 1,169 ft, are used by the city to supplement the Mill Creek supply. The maximum combined installed pump capacity of the four wells is about 7,500 gpm (11 mgd). A fifth well has been drilled but had not been put into operation at the time of this study. Well 3, the deepest of the four and the well into which water was injected, was drilled to a depth of 1,169 ft, more than 1,000 ft of which was in basalt.

Wells 1, 2, and 3, near the reservoir, pump directly into the reservoir through separate discharge pipes. Orifice plates and manometers at the ends of these discharge pipes enable the operator to determine the rate at which each well is yielding water. Well 4, which is at the eastern edge of the city, pumps directly into the city mains. The water from the wells is not chlorinated.

Suitability of Mill Creek for Recharge

There are many factors that determine whether a given water will be suitable for injection underground. Some of these factors can be evaluated; others are more difficult to determine. Even under the best of conditions, where the chemical and physical characteristics of both the ground water and the injection water are known within rather narrow limits, it is often impossible to predict how the hydraulics of a recharge system will function.

In the preliminary study made by Hart² in 1957 and in this investigation, the quality of Mill Creek water

TABLE 1
Comparative Analysis of Mill Creek Water and Well 3 Water

Constituent	Equivalents per Million—per cent	
	Mill Creek	Well 3
Calcium	51	46
Magnesium	28	29
Sodium and potassium	21	25
Bicarbonate	94	94
Sulfate	1	3
Chloride	4	2
Fluoride	1	1

westward by gravity through a 20-in. pipeline about 7 mi. to the reservoir and from there into the distribution mains. The rate of flow through the 20-in. line is controlled by gates at the settling basin.

At the reservoir, the 20-in. line terminates in a standpipe, over the top of which the water spills into a vertical annular duct leading to the reservoir. The height of the standpipe is such that a positive pressure exists in the pipeline through at least part of its length. For example, at the location of Well 3, about 5,000 ft east of the

was thoroughly investigated and an evaluation was made of its suitability for injection underground.

Chemical Compatibility

A comparison of Mill Creek water with water from Well 3 can be made by expressing the analyses of both in terms of percentage equivalents per million of total cations and total anions. For purposes of comparison, the analyses of water from Well 3 and that of the Mill Creek water sample collected Dec. 19, 1957, in percentage equivalents per million, are shown in Table 1. It is obvious from the comparison in Table 1 that both waters are of the same type, in that the dissolved solids in both predominate in alkaline earth (Ca and Mg) bicarbonates. In addition to the dissolved constituents, silica is present, equivalent to 46-42 per cent of the dissolved solids in the two waters.

Sediment

Sediment is one of the major problems when unfiltered surface water is injected directly into a well; therefore, water used for recharging ground water reservoirs should be as free from sediment as possible. Because of economic considerations, the ground water recharge program at Walla Walla was carried out with unfiltered surface water. Consequently, it was necessary to determine the magnitude of the sediment load carried by Mill Creek and to evaluate the extent to which sediment would plug the well and aquifers.

In order to determine the sediment load carried by Mill Creek at different times, 60 samples of water were taken at the city's settling basin by city personnel in April, May, and June, 1956, and 50 were taken in January, Febru-

ary, March, and December, 1957. The 110 samples, collected on separate days, were analyzed in the laboratory of USGS in Portland, Ore. The samples collected in 1956 contained 4-6 ppm of sediment. Of those collected in 1947, 44 contained 2 ppm of sediment and six contained 12-16 ppm.

During Feb. 23-28, 1957, when the latter six samples were collected, the water was not considered clear enough for public supply, and was returned to Mill Creek. The samples of December 1957, collected immediately before and during recharge, all contained 2 ppm of suspended silt and clay sediment, which represents the sediment concentration of the water injected into Well 3.

A visual check on the turbidity of the Mill Creek water is made several times each day by the operator at the settling basin. According to him, if the turbidity is great enough to obscure the bottom of the settling basin, the water can no longer be used and is then returned to Mill Creek. This degree of opacity is reached when the sediment content exceeds approximately 6 ppm.

It can be assumed, therefore, that the highest concentration of sediment in Mill Creek water that would be available for recharge is about 6 ppm. Hence, the greatest amount of sediment that could be carried into the well would be 16 lb for each acre-foot of water injected.

Air Content

When surface water is being used for artificial recharge through wells, air from two different sources can enter the aquifers tapped by the well: air carried into the well in solution in the water and air entrained in the flow of recharge water during transit.

Dissolved air. Water in contact with air will take into solution certain quantities of oxygen, nitrogen, and other atmospheric gases. The amount of these dissolved gases depends on pressure, temperature, and length of time involved. Water flowing in a stream invariably takes some air into solution. In deciding whether a surface water is suitable for artificially recharging a ground water body, consideration should be given to the amount of dissolved air in the water, inasmuch as it is a potential source of trouble. In this case the two waters were compatible since both were near saturation.

Entrained air. It is believed that the entrainment of air in the recharge water could be far more important in introducing air into the recharge system than dissolution of air. As indicated above, there is a definite upper limit on the amount of air that can be carried in solution. On the other hand, there is a much higher limit on the amount of air that can be carried along by the water column in the form of bubbles. This limit is a function of the mechanical characteristics of the recharge system.

Biologic Suitability

The water injected into Well 3 was withdrawn from the city's pipeline downstream from the point where chlorine is added for disinfection purposes. The injection of chlorine is adjusted to maintain a residual chlorine content of 0.35 ppm. This residual, although adequate to destroy pathogenic bacteria, is not sufficiently high to destroy the organisms that could cause difficulty when water is introduced underground.

To obtain information concerning the existence of such bacteria in Mill Creek water, a sample of the water

was shipped to the Sanitary Engineering Section of the Washington State Institute of Technology at Pullman. Neither odor-producing bacteria nor hydrogen sulfide-producing and iron-depositing bacteria could be isolated. Cultivation in the laboratory did produce a stringy growth, however.

The Washington State Department of Health was requested by both the city of Walla Walla and the Division of Water Resources to examine the recharge plans and to provide an opinion on whether injection of Mill Creek water underground would be practicable, from the standpoint of possible harmful effects of bacteria. The latter agency set forth the opinion that, although *Spaerotilus* (iron-producing) organisms are sometimes present in chlorinated Mill Creek water, they would not continue to flourish underground after recharge of Mill Creek water ceased.

These two reports were considered by the city to be adequate justification for minimizing possible effects of well damage or water deterioration by nuisance bacteria.

Adequacy of Supply

During part of the year the city of Walla Walla does not withdraw as much water from Mill Creek as it is entitled to, because of the seasonal variation in demand. During periods of peak demand, exceeding 14 mgd, the additional requirement is supplied by wells, and when Mill Creek water is too turbid for use, usually for only a few days at a time, ground water is substituted for the entire surface water supply. During the remainder of the year, some water from Mill Creek is available for injection into wells.

No estimate has been made of the city's water demand in future years. Probably at least 1,000 gpm would be

available for recharging during at least nine months of the year, for many years in the future. Hence, a recharge program could become a valuable part of the city's water supply system.

Yield Characteristics of Well 3

As a part of the data collection program, pumping tests were run on Well 3 to determine its yield and other hydraulic characteristics before the start of the artificial-recharge test. In all, three tests were made, two on May 28 and 29 and one on Oct. 1 and 2, 1957.

During the pumping tests of May 28 and 29, Well 3 yielded 1,800 gpm with a drawdown of 52½ ft, which represents a specific capacity (yield divided by drawdown) of 35 gpm/ft. During the test of Oct. 1 and 2, Well 3 yielded 1,630 gpm with a drawdown of 45 ft, representing a specific capacity of 36 gpm/ft. The smaller yield during the test in October resulted from a greater pumping lift, nearly 10 ft more than during the previous tests. During the period between the May and October tests, a decline of 16 ft in static level had occurred.

Recharge With Mill Creek Water

Injection of water from Mill Creek into Well 3 was started on Dec. 11, 1957, and terminated on Jan. 8, 1958, after 71.3 acre-ft had been injected. Mill Creek water entered the well through a 6-in. branch line from the 20-in. city conduit. The rate of injection was measured by means of a displacement-type meter. Some adjustment of the valve in the 6-in. line was required to regulate the rate of flow into the well. Within 30 min after the start of injection at 8:29 AM, the rate was adjusted at 630 gpm. Only slight deviation from this rate was noted during the progress of the experiment.

In 24 min, the water level rose 10.65 ft and remained within 0.03 ft of that height for at least another 22 min. From the figures 10.65 ft and 630 gpm, the specific capacity for this short period is calculated to be 59 gpm/ft, more than half again as great as the specific capacity calculated from the pumping test, which was made at a greater rate and drawdown. The difference in specific capacity probably is due to the length of time involved, and to the difference in the rates of pumping and recharging or to the extent that this difference affects the entrance loss of the well.

After a day's recharging, the water level in Well 3 rose at a fairly constant rate. Because the rate should have diminished appreciably with time, it is possible that some progressive decrease in water-receiving capacity was occurring. According to E. W. Reed,³ however, the linear relation of water level rise to time may be only coincidental, or it may result from the existence of a complex set of hydraulic boundaries within the basalt aquifers.

On Dec. 20, at 3:30 AM, recharging was discontinued, because Mill Creek became turbid as a result of storm runoff. About ½ hr before recharging was stopped, Well 1 was turned on to supply the city's demand, and ½ hr after recharging was stopped, the water level in Well 3 declined 17.3 ft. During the next 3-day period the water level declined an additional 6 ft, partly due to the pumping of Well 1.

Using 17.3 ft as the water level change and 630 gpm as the flow rate, a specific capacity of 36 gpm/ft is obtained. This value is 23 gpm/ft less than that obtained on Dec. 11, shortly after artificial recharging commenced. It is realized, however, that this apparent decrease is due in small part to the pumping of Well 1.

By Dec. 23 the flow in Mill Creek had cleared, and recharging, at a rate of 660 gpm, was resumed at 10:00 AM, 2 hr after Well 1 was shut down. After 135 min the water level had risen 17.7 ft. The indicated specific capacity at that time was 37 gpm/ft. After 16 days of recharging, the water level had risen to a level 33.2 ft above that held before the second period of recharging commenced.

On Jan. 8 recharging was stopped, which caused a drop in water level of about 24 ft. Using 670 gpm as the flow rate near the end of the test, the specific capacity was found to be 28 gpm/ft at the time the recharging was ended. The water level in the well held nearly constant for the next 3 days while the pump was idle.

As part of the recharging test, periodic water level measurements were made in Well 1. They showed that the level in the well was rising prior to the start of recharging. This may represent a residual recovery of the water level from the drawdown caused by the pumping of all three wells on Dec. 6-8. At the end of the first recharging period, on Dec. 20, the level in Well 1 had risen almost 7 ft; doubtless a substantial part of this rise can be attributed to the recharge at Well 3.

During the second recharging period, very few measurements of the water level in Well 1 were made. At the end of the second recharging period, on Jan. 8, the level in Well 1 had risen an additional 4 ft to about 105.5 ft below the measuring point.

Within 20 min after cessation of recharging, the water level in Well 1 showed a reversal and a decline. Because of outside effects, that part of the water level rise due solely to recharging cannot be identified.

A few scattered measurements in the Roy Frazier well indicate that the

water level in that well may be affected by both pumping and artificial recharge of Well 3.

Effect of Recharge on Yield

To permit evaluation of the effect of the recharging on Well 3, the well was pumped for a 48-hr period beginning at 8:00 AM on Jan. 11, 1958.

Although the static level just prior to the start of pumping was about 164 ft below the measuring point, about 9 ft higher than that before the recharge tests were run, the yield of the well was less, only 1,540 gpm, and the pumping level was 231 ft below the measuring point. The observed yield, together with the drawdown of 67 ft, represents a specific capacity of 23 gpm/ft, only about 65 per cent of the pretest average specific capacity of 35 gpm/ft. The yield was 95 per cent of that (1,630 gpm) observed during the October test.

The lower specific capacity after recharging indicates that some deterioration in yield characteristics of the well occurred as a result of the artificial-recharge experiment. At the beginning of the experiment, the specific capacity was 59 gpm/ft. As the experiment continued, the successive specific-capacity values were 36, 37, and 28 gpm/ft. The drop from 59 to 36 gpm/ft represented a 40 per cent decline during the first recharge period, when 24.3 acre-ft of water was introduced into the well. During the second period, in which 47.0 acre-ft was injected, a further decline of only 19 per cent occurred. There is evidence, then, using specific capacity as the criterion, that deterioration of the well was not linear, and that, had the recharge been continued over a longer period, less and less reduction would have accrued per unit quantity of water injected.

A water sample collected from the discharge pipe of Well 3 on Jan. 11, 1958, 30 min after pumping started, contained 76 gpm of suspended sediment. The water pumped before this time doubtless contained much more sediment. Because of the arrangement of piping, however, it was not possible to sample the first water pumped. Within $\frac{3}{4}$ hr after the start of pumping, the sediment content had dropped to 3 ppm. It is estimated that about 390 lb of sediment was carried into the well by the 71.3 acre-ft of water recharged and that somewhat less than half, or about 150 lb, was removed during the postrecharge pumping and surging period.

Well 3 was started again at 1:32 PM on Jan. 14, and then was alternately stopped and started in a surging operation. This surging was done in an effort to restore the well's prerecharge specific capacity; however, no significant increase was detected.

Recovery of Injected Water

Advantage was taken of the difference in specific conductance of Mill Creek water and native ground water from Well 3 to determine the approximate amount of injected water that was recovered during pumping of the well after completion of the recharge experiment. The specific conductance of water from Well 3 was about 169 $\mu\text{mho/cm}$ at 25°C. That of Mill Creek water ranged from 53 to 64 $\mu\text{mho/cm}$ on the basis of samples collected through the period Dec. 11–28 from a faucet on the 20-in. line a few feet from the pumphouse at Well 3.

There was a progressive trend in specific conductance, from 64 to 157 $\mu\text{mho/cm}$, in the fifteen samples collected from the discharge of Well 3 through the period Jan. 11 to Feb. 26. The specific conductance of the

first sample was only slightly more than the assumed average specific conductance of Mill Creek water as determined from the data presented. The specific conductance of a sample collected on Feb. 26 was more nearly that of native ground water. The foregoing data show that, at the beginning of the pumping period, the water pumped from Well 3 was a blend of Mill Creek water and native ground water, with a very high proportion of Mill Creek water. As pumping continued, the proportion of native ground water in the well effluent became greater and greater, until on Feb. 26 the effluent consisted largely of native ground water.

Conclusions

The basalt aquifers tapped by Well 3 accepted 71.3 acre-ft of water from Mill Creek during a 26-day period of artificial recharge. Data obtained during the recharge experiment show that some deterioration in performance of Well 3 occurred during this period; a decline both in yield and in specific capacity was noted. Three potential causes of yield deterioration have been recognized in this report: (1) sediment in the injected water, (2) dissolved air in the injected water, and (3) entrained or entrapped air that originated from leaks or undetected openings in the piping system. The relative importance of the three in causing the observed deterioration cannot be evaluated on the basis of the data now available. It is possible, of course, that the decrease in yield and in specific capacity resulted from a combination of two or more of the potential causes cited above.

The continuous blast of air emanating from the measuring port during the artificial-recharge tests suggests that large amounts of entrained air

were being carried into the well with the recharge water. This is believed true because an internal source of air in such quantity is extremely unlikely. If, during free fall of water in the pump column, water bubbles were of such size that further entrainment downward could occur within the well, many of those bubbles would enter the aquifer interstices and cause substantial plugging.

Repeated surging of the well did not improve its performance characteristics—even after almost half the estimated 390 lb of sediment carried into the aquifer with the injection water was recovered. Thus, the sediment content of the injected Mill Creek water probably was not a prime cause of plugging. Also, there seems to be no obvious way for the dissolved air to impair the well or aquifer, inasmuch as it is not likely to be released from solution. It is true, of course, that oxidation of ferrous hydroxide would be a possible cause of aquifer plugging, if enough ferrous iron were present to yield large quantities of precipitate. The analytical data show that the amount of iron in solution in native ground water from Well 3 is extremely small, however.

On the other hand, the sediment remaining in the well or in the aquifer immediately adjacent to the well may have an effect far more deleterious than might be supposed from the small amount of sediment involved. Furthermore, oxidation of ferrous iron present in the ground water by the air entering the aquifer may have a much greater effect than is now suspected.

The air entrapped in the Mill Creek water could be removed by passing the water through deaeration equipment prior to its injection into the well. Use of a regulator foot valve at the

bottom of the pump column would prevent the free fall of water and would virtually eliminate the problem of air leaving solution above the water level in the pump column. Filter beds could be used to remove the sediment prior to injection of the water into the well.

The decision to continue experiment at Well 3 must be based on a realistic appraisal of all pertinent factors. The inherent risk of ultimate permanent damage to the well could conceivably be a strong deterrent to the continuation of the experiments. There are, however, three important factors that should be considered in arriving at a decision:

1. Periodic redevelopment of the well by the use of chemical agents and other techniques might become an integral part of any long-term program of recharge. A chemical washing agent proved satisfactory in the redevelopment of an injection well in the Grand Prairie region of Arkansas.⁴ Although the aquifers involved are lithologically different, a similar redevelopment procedure might be employed at Well 3 in the event that considerable loss in yield occurred.

2. A definite need for recharge exists in the general area of Wells 1, 2, and 3. Water level gradients here seem to be consistently low, one cause of which could be the existence of the hydraulic boundaries detected during the recharging experiment. These boundaries may limit the natural recharge of this area. It is likely, therefore, that the long-term small water level decline, as indicated by the series of measurements begun in the early 1940's, indicates local overdraft. If an overdraft is occurring, water will have to be pumped from increasingly greater depths with consequent increased pumping costs. Artificial recharge

could be used in order to augment the supply.

If the air entrapment and sediment problem can be resolved, as described above, alternate recharging and pumping of Well 3 could be tried, as this procedure might help sustain the highest possible specific capacities. Recharge water might be injected into the well for a selected period of time, and then the well could be pumped for a short while. Experimentation of this type probably would determine the most effective periods of alternate recharge and pumping. For example, it is suggested that recharge water be injected for about 2 days, and that the well be pumped for about 2 hr to determine any change in yield and draw-down. If no appreciable deterioration has occurred, a longer period of recharging could be tried.

3. The water system of the city of Walla Walla is ideally arranged to supply large quantities of water from Mill Creek for artificial recharge with a minimum of attention. During the test, 71.3 acre-ft (23.2 mil gal) was injected in Well 3 at virtually no cost, except that of the installation of the crossover piping. Because of the

readily available supply of Mill Creek water for recharging and because of the definite gain that would be derived from a recharge program, provided that a technique could be developed having no serious adverse effect on well or aquifer, continuation of the experiments would seem to be worthwhile.

Acknowledgment

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Production Analysis of Artesian Wells

A. L. Simon

A contribution to the Journal by A. L. Simon, Research Asst., School of Civ. Eng., Purdue Univ., Lafayette, Ind.

THE importance of ground waters is greater than ever. As surface waters are being more and more polluted and contaminated, the ground water demand is constantly increasing. Artesian water supply is not unlimited. In almost every civilized country where the water supply is derived largely from artesian wells, artesian heads are generally decreasing. This means not only an increase in pumping expenses, but also a loss of usable ground water. It should be borne in mind that this loss is largely due to the interconnection and recharge of aquifers by improperly constructed wells—not to proper ground water development. The waste of water and energy can be eliminated by proper investigation.

Discharge-Drawdown Analysis

The relationship of available water to drawdown is determined by means of pumping tests. The results of such tests can be graphically represented as so-called " $Q-s$ " curves, in which Q represents the discharge in gallons per minute and s represents the drawdown in feet from the static water level.

According to Darcy,² the discharge-drawdown curve of an artesian aquifer is a straight line. This law has two limitations. In the region where the flow velocities are increased and inertia effects become important, the head loss

rate gradually increases. For this reason, the rate of discharge with respect to drawdown becomes less and less and, finally, the well ceases to give more water even at an increased drawdown.

The other limitation of Darcy's law is given by the theory of active cross sections. According to this theory, the area of open (active) pores in the pervious medium varies according to the hydraulic gradient, the average grain diameter of the sand, and mainly to the total hydrostatic pressure on the aquifer. In artesian conditions there is a limit gradient—the so-called threshold gradient. At this hydraulic gradient there are no available open pore areas in the porous medium, hence there is no flow. This threshold gradient is obtained by the equation of Juhasz,⁴ which is written:

$$J_0 = \frac{1.88\alpha}{n^4 r_0} P_r$$

in which J_0 is the threshold gradient, α is 3×10^{-11} coefficient of static friction for water, n is the porosity, r_0 is the effective radius of a sand particle (10 per cent), and P_r is the total hydrostatic pressure on the aquifer. If one takes the threshold gradient into account, Darcy's law may be written in the form

$$v = \kappa [J - J_0]$$

in which v is the velocity, κ is the permeability, and J is the hydraulic gradient.

Under normal conditions, the hydrostatic pressure, the effective sand diameter, and the porosity vary along the thickness of the aquifer. Hence the threshold gradient for a natural aquifer is not a definite value. Its effect, however, shows up on the discharge-drawdown curve in almost every case.

On account of the preceding factors, one may conclude that the ideal discharge-drawdown curve for artesian aquifers may be plotted as shown in Fig. 1. Data taken from several thousand artesian wells in Hungary agree with this ideal curve. These wells are located in areas with ideal artesian aquifer conditions, and each produces from one aquifer only.

Deviations from the ideal discharge-drawdown curve are due to stratification of the aquifer; large aquifer thickness; limited water storage, and interconnection of several aquifers by lack of cementation, improper sealing, or by the well itself. Because the interconnection of aquifers is the main cause of deviation, it is discussed in some detail below.

Interconnected Aquifers

Most of the deeper artesian wells pump from two or more aquifers. The different aquifers differ in their characteristics, such as the elevation of the static water level or the shape of the discharge-drawdown curve. Data from common drilling practice do not reveal these characteristics, however. Usually, information concerning the static water level of the well and the well's discharge-drawdown curve is supplied to the driller in ad-

vance. Both of these characteristics indicate the nature of the different aquifers. Quite often, measurement of each aquifer's characteristics during the drilling procedure is uneconomical. There are several methods, however, by which one may obtain these characteristics later if they are required.

The static head of a well that interconnects two aquifers is at an intermediate elevation between the highest and the lowest static levels of the inter-

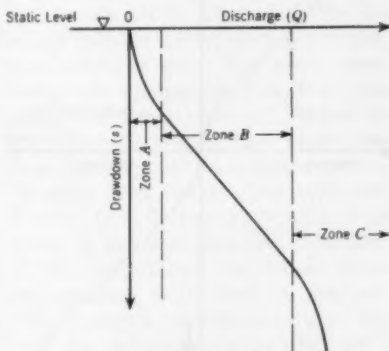


Fig. 1. Generalized Ideal Discharge-Drawdown ($Q-s$) Curve

Zones A, B, and C are the zone of influence of the threshold gradient, the zone of validity of Darcy's law, and the zone of influence of inertia and turbulent effects, respectively.

connected aquifers. To investigate it properly, a slight improvement should be made on the ideal discharge curve. Theoretically for well hydraulics and field work, the recharge of any well may be computed the same way as the discharge. In other words, the shape of the discharge-drawdown curve is the same as the shape of the recharge-recovery curve rotated 180 deg on the $Q-s$ coordinate system (Fig. 2).

Interconnection between two aquifers creates a static level different from that of either aquifer. Hence the pressures on the aquifers are not in equilibrium with the pressures in the aquifers. This causes a discharge from



Fig. 2. Recharge-Discharge Curve

The recharge-recovery curve is theoretically a mirror image of the discharge-drawdown curve.

the aquifer of higher static level to the aquifer of lower static level. If, for example, the drawdown level of the well being pumped is beneath the static water level of one of the aquifers, this aquifer does not produce but decreases the amount of water pumped by recharging. These relations can easily be investigated by a simple graphical method. This may best be understood

by considering the easier case, in which the discharge-drawdown curves and the static levels of both aquifers are known. To obtain the static level of the well and the resultant curve, one may plot a graph showing the ground surface, the static water level of both aquifers, and the discharge and recharge curves for both aquifers (Fig. 3).

In Fig. 3, the characteristics of two aquifers are shown. The well characteristics, such as its cumulative discharge-drawdown curve and the static water level of the well, can be obtained by graphical addition. The static head in the well should be at the elevation where the resultant curve crosses the vertical axis—in other words, at the level where the discharge of the upper aquifer is equal to the recharge of the lower aquifer. Two more check points should be recognized on this graph: the cumulative curve should cross the discharge or recharge line of an aquifer at the elevation of static head of the other aquifer. This is because at these points the other aquifer does not contribute to the total discharge.

Figure 3 furnishes some more important details about the hydraulics of the well. One may, for example, determine quantitatively whether the well would supply more water if the lower layer were closed off—provided, of course, that the water demand is such that the upper layer can supply all of it alone. If so, the static water level of the well may be increased considerably, and the need for pumping installations may be eliminated. In some cases, a poor aquifer of higher static level alone would give better results than a very good aquifer of low static level. It must be recognized that, if the former has a very "good" recharge

capacity, it is less advantageous to pump more water from a greater depth than to pump less water from a much lesser depth. There are numerous other questions that may be answered by the graphical method. It should be pointed out that the higher static level does not necessarily represent a higher

aquifer. Two such generalized curves are shown in Fig. 4. The difference between them is very important in the production evaluation.

Rheometer Tests

In practice little is known about a well other than its static level and discharge-drawdown curve. If there is more than one aquifer, the characteristics for each may be obtained by rheometering. A rheometer is a propeller-driven flowmeter. First, the spinner should be lowered to the top of the deepest layer; then a pumping test should be run. For every drawdown, the amount of flow from this aquifer should be obtained. When the drawdown at which there is no flow from the aquifer has been reached, its static level has also been reached. If this $Q = 0$ drawdown cannot be found, it indicates that the static level of the investigated aquifer is above the resultant static level of the well. The aquifer's discharge curve can then be subtracted from the well's cumulative-discharge curve. The same procedure should be followed when the spinner is at the top of the second aquifer. The discharge curve obtained by the rheometer is a cumulative curve for the two lowest aquifers. By graphical subtraction, one can obtain the static levels and discharge-drawdown curves for the two aquifers. By repeating the same procedure, one can obtain the characteristics for each aquifer separately. This method makes possible a proper discharge analysis of the well.

External Interconnection

An important problem arises when two aquifers of very different static water levels are interconnected, not

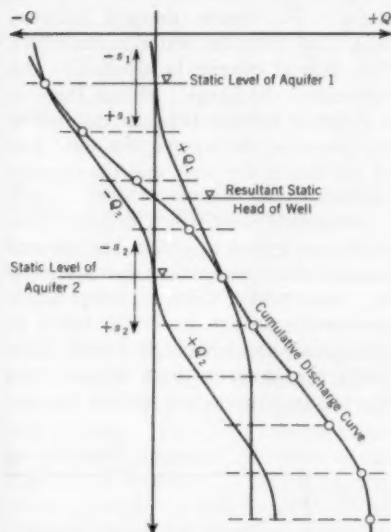


Fig. 3. Construction of a Cumulative Discharge Curve for Two Interconnected Aquifers

The cumulative discharge curve is plotted by graphical addition of the discharge values of the two aquifers at various drawdowns.

or lower aquifer location. The relative depth of the layers has no importance in this investigation.

In artesian wells, different aquifers may be compared according to the slope of their discharge-drawdown curves. The slope is a linear function of the permeability and thickness of the

through the well, but outside the casing. This is a construction failure caused by lack of cementation or improper sealing. The head and discharge losses can be extensive. This type of trouble is very difficult to detect. Several methods, such as radioactive tracing or temperature anomaly measurement, can be used to detect it. The existence of such a condition may be indicated by an excessive deviation from the ideal discharge-drawdown curve for the particular aquifer. This deviation arises because the actual curve is the resultant of the curves for two interconnected aquifers.

Thermal Dilatation

It is commonly known that the temperature of the earth gradually increases with depth. The temperature variation, known as the thermal gradient and expressed in degrees per foot, varies according to the geographic region.

Temperature measurements in a nonproducing well under construction reveal the average thermal gradients of the underlying formations. Figure 5 shows the information obtained by such measurements.

If the well is producing, the water flowing up the well will be at a higher temperature than the surrounding rocks. To regain thermal balance, heat flows from the water to the rocks. This flow of calories is supplied by the continuous discharge. Hence there is a dynamic balance between calories of the water in the aquifer, the total loss of calories in the well, and the calories of discharging water.

In certain cases, when deep artesian wells are drilled to obtain hot mineral waters, this temperature decrease may be important. Single temperature measurements are generally taken at the surface and bottom of a well. The difference between these values gives the fall in temperature during the up-

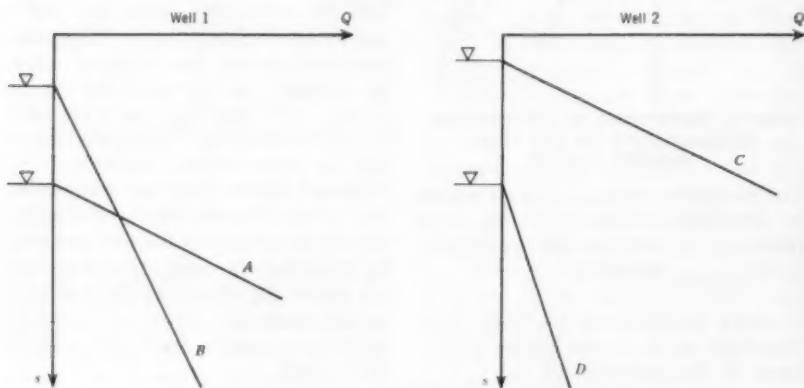


Fig. 4. Comparison of Discharge Characteristics of Two Aquifers

The slopes of the generalized curves have the following significance: Curves A and D, low static head, good discharge characteristics; Curves B and C, high static head, poor discharge characteristics.

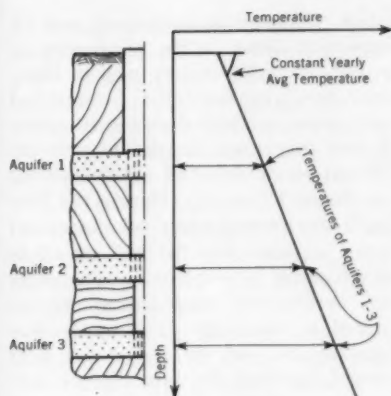


Fig. 5. Measurement of Thermal Gradients in a Well During Construction

The thermal gradients of the penetrated layers are measured during the construction of the well.

ward flow. The decrease in temperature divided by the discharge gives the rate of heat loss in calories per unit of time. This indicates that the temperature decrease decreases with a discharge increase. Analysis of this phenomenon shows that the decrease in temperature of the produced water is a function of the depth of well, the amount of discharge, and the thermal conductivity of layers, as well as other factors.

Figure 6 shows the influence of discharge on the temperature of a well. If $Q = \infty$, there is an infinite supply of heat—hence no decrease in temperature. If $Q = 0$, the water cools down to the temperature of the surrounding rocks.

In reality the temperature curve is not linear; however, it is assumed to be linear to simplify computation. The average temperature of the artesian head can be computed by taking

half of the sum of the temperatures at the bottom and at the head of the well. This average temperature is a function of the depth and the discharge rate and is therefore variable. Because the density of water varies with temperature, the unit weight of water in a well is a function of the depth and the discharge rate.

Effect of Temperature on Head

The artesian head H should counterbalance the pressure P_o in the well

$$P_o = H\gamma_w$$

If the γ_w decreases, H should increase to get the same product. Hence, it may be concluded that the static artesian heads are not constant but are dependent on certain conditions in the well, such as its depth and discharge rate.

The average density of water in a main producing well is approximately

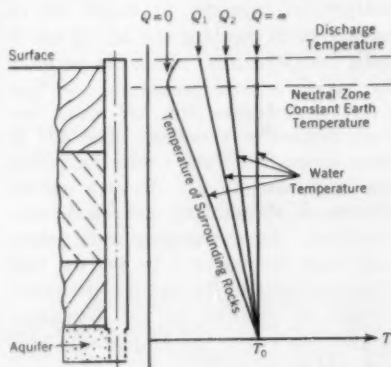


Fig. 6. Temperature Changes of Water as a Function of Discharge

In reality the temperature curves are not linear; they are assumed to be linear to simplify computations.

1.012, if the T_o is 90°C and surface temperature (yearly average) is 15°C . If the same well is pumped so that the water temperature at the surface is 80°C , then the density will change to 1.0324. If the head of the well was originally 1,000 ft, the head increase (ΔH) due to temperature change will be $(1.0324 - 1.0120) \times 1,000 = 20.4$ ft. The new head will be 1,020.4 ft. This change in head considerably increases the discharge of the well. A temperature increase also affects the viscosity of water, which, in turn, decreases the head loss in the casing and increases the discharge. A graph having the characteristic shape of that in Fig. 7 may be constructed for any well, showing the change of head with respect to discharge. Such a graph can be used in connection with a graphical analysis of the discharge. It should be remembered, however, that the new head will finally be established only after a time delay.

When continuous pumping or free outflow is expected, it might be of advantage to combine the ΔH - Q curve with the Q - s curve. This can be done by taking several points on the Q - s curve and finding the ΔH value for each respective Q value. The ΔH is then subtracted from s and the points are connected to get the new curve. Figure 8 shows this graphical construction. Long pumping tests generally yield this curve. To get the real characteristics of the aquifer, the curve should be reduced to the discharge-drawdown curve by graphically adding ΔH to s .

When the effect of the thermal dilatation of artesian heads is taken into account, the characteristic discharge curve improves. The author witnessed a case where the effect of thermal dilata-

tion, although unrecognized, was an important factor in the production of a well. A 1,300-ft deep well in Hungary, supplying hot (60°C) water, had continuous artesian flow for 10 years. A new magistrate decided to save the effluent water and had a valve placed on the well's outlet. During the first night, the resting water column cooled down so much that the well ceased to overflow the next morning. The original overflowing head had to be restored by pumping. The reason was unexplained and the valve was kept closed, but after the third time of clos-

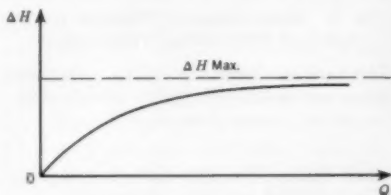


Fig. 7. Relationship of Change of Head to Discharge

The curve is asymptotic with a theoretical limit for ΔH , reached when discharge (Q) reaches infinity.

ing and pumping, the magistrate decided to let the well overflow.

The first commercial application of thermal dilatation was made in 1953 at Nagylengyel Oil Field, Hungary, where a large amount of pumping and the installation of a power supply were avoided by the excessive increase in head and discharge.

Corrosion

In the production analysis of artesian wells, it is generally believed that the discharge characteristics of a well will change but slightly over a period of time. The slight change is due to

the loss of artesian water and is a general property of the particular ground water region. A more important factor is the sudden change in discharge characteristics. This is largely due to the perforation of the casing by corrosion. In order to return the well to normal operation, leaks must be found and repaired. Leaks are easily found by rheometry, and closing these outlets is a simple technological procedure.

In many cases, however, losses due to leakage in a well casing may never be noticed. As a result, there is a possibility of well contamination by polluted surface waters. It is the duty of the engineer to prevent such possibilities by corrosion protection.

The most common indication of corrosion is ferrous water. The usual procedure used to correct this is de-ironing, which, in most cases, is entirely wrong. To understand the problem, one must be familiar with the causes of well corrosion. There are two kinds of well corrosion: external and internal. External corrosion is uncommon and its occurrence is confined to certain depths. Internal corrosion is frequent and occurs along the whole length of the casing. Internal corrosion is the dissolution of the steel casing by aggressive materials originating in the aquifer. The dissolved iron makes the water being discharged ferrous.

Experience shows that the iron content of few artesian waters comes from the aquifer. The geology of the region determines the likelihood of ferrous deposits. Water samples taken from the bottom of the well can decide this question.

The most important aggressive materials in the artesian waters are hy-

drogen sulfide and carbon dioxide. Both of these are either created by oxidation of organic materials in rocks or by postvolcanic effects and are commonly found in aquifers. Of the 30,000 Hungarian artesian wells 65 per cent contain enough carbon dioxide to be aggressive to iron. The carbon dioxide content can be extremely large; indeed, there are wells in Mihalyi, Hungary, producing nothing but carbon dioxide.

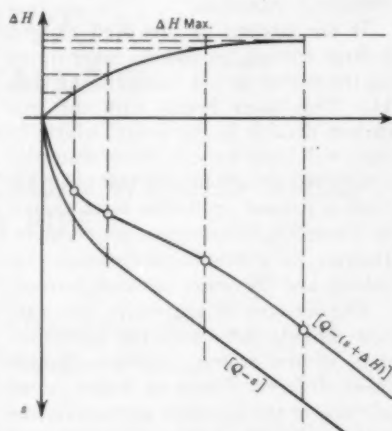


Fig. 8. Combination of ΔH - Q and Q - s Curves

As can be seen, consideration of the effect of thermal dilatation improves the discharge curve.

The chemical relationships between the water, iron, and aggressive materials are very complex. There are three possible chemical effects on the casing:

1. A thick calcium carbonate layer may be deposited on the casing and clog up the upper part of the casing, sometimes within a few months.

2. A thin natural protective layer may be deposited on the casing by iron and calcium carbonate and remain there unaltered.

3. The casing may be gradually eroded away.

The solubility of certain minerals in a well water decreases with upward flow because of the drop in temperature and pressure. This may cause the continuous deposition of calcium carbonate. If so, the layer should frequently be removed by drilling or by ultrasonic vibration.

If the amount of dissolved oxygen is large enough, an alkaline layer forms on the casing at pH values as high as 11. This layer bonds with the free carbon dioxide in the water, which, in turn, will bond the calcareous materials if they are present in sufficient quantity. Such a natural protective layer cannot be formed in soft, oxygen-poor waters. Instead, the casing is continuously dissolved, and the water becomes ferrous.

The amount of aggressive free carbon dioxide influences the corrosiveness of the water. Carbon dioxide takes different forms in water. One of these is the so-called aggressive free carbon dioxide. The percentage of free carbon dioxide depends on the pressure and temperature. In upflowing artesian water the release of aggressive free carbon dioxide is uniform, and this makes the corrosion uniform along the casing.

At the upper section of the well, some of the aggressive chemicals are transformed into gas. In this form, their corrosive power is usually higher.

Thus the first leakage of a well casing is most likely to happen in the upper part of the well.

Corrosion Protection

There are several methods of corrosion protection, such as the use of sleeve pipes, the application of protective paint coats, chemical and electrochemical methods for creating protective layers, and cathodic protection. Each method has its advantages and disadvantages. Many variables must be taken into account when evaluating them.

By applying corrosion protection, one may decrease the iron content of the water and de-ironing will not be needed. On the other hand, the free carbon dioxide should be driven out of the water before distribution. This saves the distributing network from corrosion and prevents the growth of iron bacteria.

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AWWA C201-60T

American Water Works Association

TENTATIVE

AWWA STANDARD

for

**FABRICATED ELECTRICALLY WELDED STEEL
WATER PIPE**

This "Standard for Fabricated Electrically Welded Steel Water Pipe" is based upon the best known experience and is intended for use under normal conditions. It is not designed for unqualified use under all conditions and the advisability of its use for any installation must be subjected to review by the engineer responsible.

First Edition approved Apr. 25, 1940; Second Edition approved as "Tentative" May 15, 1960.

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Acknowledgment is made and appreciation is expressed to the Subcommittee on Pipe of the Steel Water Pipe Manufacturers Technical Advisory Committee for its assistance in the preparation of this standard. The officers of this committee are R. C. Beam, chairman; Walter H. Cates, secretary.

Latest Revisions to C201

This Second Edition of AWWA Standard C201 was approved as "Tentative" on May 15, 1960. The principal change involved is that this edition covers fabricated electrically welded pipe of all sizes, whereas the First Edition covered only sizes 30 in. and larger.

Designation: As a result of the revision, the former designation, "C201-59," has been changed to "C201-60T."

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AWWA C201-60T

Tentative AWWA Standard for

**Fabricated Electrically Welded Steel
Water Pipe**

Section 1—General

Sec. 1.1—Scope

This standard covers the manufacture of fabricated straight butt seam or spiral butt seam electrically welded steel pipe intended for the conveyance of water, or other special sections which are required for closures, curves, or changes in alignment or grade to meet special conditions. For the purpose of this standard, fabricated pipe shall be defined as steel pipe fabricated from plates or sheets; the properties of the material are determined prior to fabrication.

Sec. 1.2—Definitions

Under this standard the following definitions shall apply:

1.2.1. *Purchaser*: A person, firm, corporation, or governmental subdivision entering into a contract or agreement for the purchase of any materials or for any work to be performed.

1.2.2. *Contractor*: The person, firm, or corporation executing the contract or agreement with the purchaser to furnish any material or perform any work.

1.2.3. *Inspector*: The inspector or those assistants of the purchaser prop-

erly authorized and limited by the particular duties entrusted to them.

1.2.4. *Fusion welding*: The process of welding metals in the molten, or molten and vapor, state without the application of mechanical pressure or blows.

1.2.5. *Root*: The zone at the bottom of the cross-sectional space provided to contain a fusion weld.

1.2.6. *Throat*: The thickness of weld along a straight line passing through the root, excluding weld metal above plate surface.

1.2.7. *Fillet weld*: A weld of approximately triangular cross section which lies in a plane disposed approximately 45 deg with respect to the surfaces of the parts joined. The size of a fillet weld shall be expressed in terms of the width in inches of one of its adjacent fused legs, the shorter if unequal.

1.2.8. *Butt weld*: A weld whose throat lies in a plane disposed approximately 90 deg with respect to the surface of at least one of the parts joined. The size of a butt weld shall be expressed in terms of its net throat dimensions in inches, excluding weld metal above plate surface. A double-welded butt joint is one in which the

filler metal is added to both sides. A single-welded butt joint is one in which the filler metal is added to one side only.

1.2.9. *Flame cutting*: The process of severing metal by means of a gas flame.

1.2.10. *Nominal wall thickness*: The named or given thickness as distinguished from the actual or measured thickness.

1.2.11. *Spiral-seam fusion-butt-welded pipe*: Pipe in which the line of the fusion seam is a helix on the barrel of the pipe.

1.2.12. *Straight-seam fusion-butt-welded pipe*: Pipe in which the line of the fusion seam parallels the axis of the pipe.

1.2.13. *Resistance-welded pipe*: Pipe made by a series of operations in which the flat-rolled material is cold shaped into tubular form, and the union of the seam effected by the application of heat and pressure. The welding heat is generated at the seam by resistance to the flow of an electric current in a circuit of which the pipe to be welded forms a part.

1.2.14. *Ladle analysis*: The chemical analysis taken at the time the steel is being cast into ingots.

1.2.15. *Check analysis*: The chemical analysis taken from the skelp or plate.

1.2.16. *Coupon (weld coupon)*: A pair of test plates of specified dimensions made from metal of the same specifications and thickness as the pipe wall, prepared for welding, and attached to a pipe shell being welded at the end of a longitudinal joint so that the edges to be welded in the test plates are a continuation of, and exactly similar to, the corresponding edges of the longitudinal joint. The pair of plates is then made into a single weld test specimen by having the weld

metal deposited in the test plates continuously with the weld metal deposited in the longitudinal joint by the same technique.

1.2.17. *Girth seam*: A circumferential welded seam lying in one plane, used to join lengths of straight pipe or to join pieces of mitered pipe to form fabricated elbows or turns.

1.2.18. *Grade of material*: The designation given to a material according to its chemical analysis or physical properties.

Sec. 1.3—Supplementary Details to Be Specified by Purchaser

When purchasing pipe under this standard, the purchaser shall provide specific supplementary information regarding the following details:

1.3.1. Nominal inside or outside diameter of pipe.

1.3.2. Thickness of steel plate or sheet material.

1.3.3. Laying length of pipe sections.

1.3.4. Working water pressure for each thickness of pipe.

1.3.5. Hydrostatic test pressure for fittings.

1.3.6. The maximum number of longitudinal and girth seams for each pipe section.

1.3.7. Type of field joints (description or drawings).

1.3.8. Standard designation and grade of steel material required, as set forth under Sec. 2.1.1, or as otherwise specified.

1.3.9. All special sections, indicating for each component part the dimensions, the standard designation and grade of material required, the type of ends, and the required hydrostatic test pressure.

1.3.10. The drawings that are to be furnished by the manufacturer.

1.3.11. Qualification code for welders, if other than that set forth under Sec. 3.5.3.

Sec. 1.4—Inspection

1.4.1. All work done and material furnished may be inspected by the purchaser during all phases of manufacture and testing, but such inspection shall not relieve the contractor of his responsibility to furnish material and perform work in accordance with this standard.

1.4.2. The contractor shall notify the purchaser in advance of the production and fabrication of materials in order that the purchaser may, if desired, arrange for mill and shop inspection.

1.4.3. The inspector shall have free access to those parts of all plants that are concerned with the furnishing of materials or the performance of work. The contractor shall furnish the inspector reasonable facilities and space without charge for the inspection, testing, and obtaining of any information he desires concerning the character of material used and the progress and manner of the work.

1.4.4. The inspector may reject any or all pipe sections or special sections that are not of the material specified, that are not fabricated in accordance with the outlined procedure, or that do not conform to the prescribed test results, condition of pipe, and tolerances set forth in this standard, the approved drawings, and the purchaser's supplementary specifications.

Sec. 1.5—Materials and Workmanship

1.5.1. All materials furnished by the contractor shall be of the specified quality. All work shall be done in a thorough workmanlike manner by mechanics skilled in their various trades.

All material furnished and all work done shall be in accordance with this standard.

1.5.2. Any plate, sheet, or fabricated pipe that reveals scars or injuries from hammering or other causes may be rejected. Any plate or sheet that develops irreparable defects during fabrication of pipe will be rejected. If structural defects, pits, or laminations are found in any material and, in the opinion of the inspector, cannot be corrected satisfactorily, the material will be rejected.

Sec. 1.6—Equipment

1.6.1. *General.* Manufacturer's equipment, such as for welding, flame cutting, or other operations, shall be of a standard and quality necessary to produce pipe meeting this standard. Equipment, in general, shall be in good repair and shall be modern, as judged by the standards of the industry.

1.6.2. *Arc welding equipment.* Generators and transformers shall be designed expressly for welding, shall be capable of delivering substantially constant current, and shall be adjustable through a range ample for the requirements of the work. They shall be provided with controlling devices and suitable meters showing the welding current and arc voltage at all times. Automatic welding heads with suitable auxiliary handling equipment to provide control of electrode speed and welding arc shall be subject to the requirements of this standard. Welding cable shall have sufficient conductivity to avoid inadequate voltage and current at the arc and shall be insulated sufficiently to avoid short circuits. Electrode holders shall provide clean surfaces in contact with electrodes and shall hold electrodes firmly as required by the welding process.

Sec. 1.7—Drawings

Drawings shall be the purchaser's drawings or the contractor's drawings approved by the purchaser.

Sec. 1.8—Testing

Pipe sections and special sections shall be tested as required herein. Steel material shall be subjected to

the required mill test at the steel producer's plant or, at the contractor's option, a test of a specimen plate of the steel material will be made as stated in this standard. Tests of welding operators, tests of welded joints, and the hydrostatic test of finished sections shall be made as stated under Sec. 5 of this standard.

Section 2—Description of Pipe

Sec. 2.1—Steel

2.1.1. *Steel plate.* Steel plate shall be of Grades B, C, or D of ASTM A283 (Low and Intermediate Tensile Strengths of Carbon Steel Plate for Structural Quality), or as otherwise specified by the purchaser. Steel plate shall be ordered to a thickness with a maximum allowable variation of 0.01 in. less than the specified thickness. The inspector may witness manufacture and tests of plate at the mill and may require that the contractor furnish him test reports on each heat from which the plates are rolled.

2.1.2. *Specimen plates.* If requested by the purchaser, the contractor shall furnish specimen plates of the steel that will be used in the manufacture of the pipe. Specimen plates shall be of the size necessary to obtain the number of specimens required in the plate specifications, and the tests of the specimens will be made by the purchaser. If the specimens fail to meet the requirements of the specifications under which they were manufactured, a second test shall be made, and if all the specimens meet the requirements, the plate will be accepted, but if any of the specimens fail to meet the requirements, all plate furnished under the same heat number will be rejected. The contractor shall re-

imburse the purchaser for all expenses incurred in making a second test.

2.1.3. *Steel sheets and coils.* Steel sheets or coils used shall conform to the physical properties of Grade A material specified in the latest revision of ASTM A245 (Light-Gage Structural-Quality Flat Hot-Rolled Carbon Steel), or as otherwise specified by the purchaser.

Sec. 2.2—Pipe Dimensions

2.2.1. *Diameter.* The pipe diameter shall be the nominal inside or outside diameter as specified by the purchaser.

2.2.2. *Length of sections.* All straight pipe sections shall be of the length specified by the purchaser. The purchaser, in his request for bids, shall have the right to specify shorter lengths, which may be required for closures and proper location of any special section.

Sec. 2.3—Ends of Sections

Ends of pipe sections supplied under this standard shall be of the type specified by the purchaser in his request for bids. Available types are:

a. Ends for mechanically coupled field joints

b. Ends with slip joints (lap joints) for field welding

- c. Plain ends or beveled ends for field butt welding
- d. Ends fitted with butt straps for field welding
- e. Ends for riveted field joints
- f. Bell-and-spigot ends with rubber gasket
- g. Plain ends fitted with flanges
- h. Taper ends for driven field joints.

Sec. 2.4—Seams

Longitudinal seams, spiral seams, and shop girth seams shall all be butt welded.

Sec. 2.5—Special Sections

Curves, tees, branches, and other sections required for changes of alignment or grade which are made up of

sections of straight pipe shall have the required type of ends, shall be manufactured from steel plate or sheet, cut, beveled, and formed to the required shape, or fabricated in accordance with the contractor's approved drawing. When such details are not specified, the angular change for welded plate specials shall not exceed $22\frac{1}{2}$ deg per cut for curves or 45 deg for branches and wyres. Saddles, nozzles, or other fittings intended for connection to other pipelines and structures shall be of the required material and fabricated in accordance with detailed description or drawings furnished by the purchaser; or they shall be fabricated in accordance with the contractor's approved drawings.

Section 3—Fabrication of Pipe

Sec. 3.1—Preparation of Edges

Edges of plates and sheets to be joined by automatic welding shall be cut mechanically to the shape required for the particular welding process. Edges of plates or sheets to be joined by manual welding shall be formed to the shape required by the purchaser or in accordance with the contractor's approved drawing. The resulting edges for welding shall be uniform throughout the length of the plate or sheet and shall form a straight line.

Sec. 3.2—Lap Breaking and Forming

Before the forming of longitudinal edges, all plates or sheets shall be lap broken by a continuous rolling operation or in a press having dies that are machined to the proper radius of the pipe shell. The pressure exerted in the lap-breaking operation shall be sufficient to secure a true and uniform curve at the edges. Plates or sheets shall be formed to a true cylinder.

Plates formed so as to require hammering in order to obtain the desired shape will not be accepted. After forming, all pipe surfaces shall be inspected for pits or laminations. The inspector may reject any defective plate or sheet or permit its resubmittal for inspection after the use of an approved method of reconditioning.

Sec. 3.3—Cleaning of Surfaces to Be Welded

All plate or sheet edges to be welded and surfaces adjacent to them shall be free of oil, grease, and rust.

Sec. 3.4—Fitting Up

No plate or sheet shall be fitted up until it has been approved by the inspector. The plates or sheets to be welded shall be accurately matched and retained in position during the welding operation. Tack welds may be used to hold the edges in line, provided the tack weld thickness will permit the

tack weld to be thoroughly fused with and become a part of the strength weld in the welding operation. The method employed shall produce a finished pipe having a circular cross section throughout its length.

Sec. 3.5—Requirements for Welding Operations

3.5.1. The contractor shall be responsible for the quality of all work performed by his welding organization.

3.5.2. All welds produced on automatic welding machines shall meet the test requirements of this standard but this shall not be construed to mean that automatic-machine operators shall be qualified as manual-welding operators.

3.5.3. Manual-welding operators shall be qualified under Section IX, Part A, of the ASME Boiler and Pressure Vessel Code, the standard qualification procedure of the American Welding Society or any other code specified by the purchaser.

3.5.4. The inspector shall have the right at any time to call for and witness the making of test specimens by any welding operator in accordance with this standard, and the expense of such tests shall be borne by the contractor.

Sec. 3.6—Welding

3.6.1. Welding of the following types shall be permitted and shall be considered equal alternatives: fusion butt welding of straight or spiral seams, resistance butt welding of straight or spiral seams, and induction butt welding.

3.6.2. Plates shall be held in the correct position. Abutting edges shall be properly prepared. Each deposited layer of welded metal in the fusion welding process shall be thoroughly

cleaned before additional weld metal is applied to its surface. Finished weld bead shall be central to the seam and the finished joint shall be free from depressions, undercut edges, burrs, irregularities, and valleys. The inner surfaces of the pipe shall be free from burrs and other irregularities resulting from welding other than the normal bead necessary.

3.6.3. All welds shall have complete fusion with base metal and shall be free from cracks, oxides, slag inclusions, and gas pockets. If the automatic welding machine does not obtain a weld that will penetrate through to the inside of the pipe and protrude beyond the contour of the surface, an inside pass shall be made in the root of the groove on the inside of the pipe. Chipping out of the weld in the root of the groove will be required when deemed necessary by the inspector.

3.6.4. If welding is stopped for any reason, special care shall be taken when welding is resumed to obtain complete penetration between weld metal, plate, sheet, and weld metal previously deposited, and if flux is used it must be redistributed before work is resumed.

Sec. 3.7—Corrections to Welds

3.7.1. Welds found deficient in dimensions but not in quality shall be enlarged by additional welding after thorough cleaning of the welds and adjoining plate.

3.7.2. Welds considered by the inspector to be deficient in quality or made contrary to any mandatory provision of this standard shall be removed by chipping or melting and shall be remade.

3.7.3. In the removal of part or all of a weld by cutting or chipping, such cutting or chipping shall not extend

into the base metal beyond the depth of weld penetration. In the removal of part or all of a weld, care shall be taken not to burn or otherwise injure the base metal. After this operation the burnt metal shall be completely removed to clean, sound metal in preparation for rewelding.

Sec. 3.8—Automatic Welding

All longitudinal, spiral, and girth seams of straight pipe sections, and special sections when practicable, shall be welded with an automatic welding machine. Sample welds shall be submitted to the inspector for testing. Approval of such tests shall be required prior to welding of the pipe.

Sec. 3.9—Manual Welding

Manual welding of girth seams will be permitted on extraordinarily long straight pipe sections, consisting of more than one standard length, when agreed to by the purchaser and contractor. Manual welding of special sections and fittings will be permitted when it is impractical to use an automatic welding machine. On straight pipe sections the only manual welding permitted shall be that required in tack welding of coils and plates during the continuous pipe-making process, in making an inside pass on the inside of the pipe, in rewelding and repairing structural defects in the plate and automatic machine welds, and as otherwise permitted in this standard.

Sec. 3.10—Rounding of Sections and Sizing of Ends

If it is necessary to reshape pipe after it has been welded, reshaping shall be performed by rerolling or by pressure. Reshaping of pipe by dropping or by hammering will not be permitted. Sizing of pipe ends to

come within specified end tolerances shall be permitted.

Sec. 3.11—Preparation of Ends

3.11.1. *Ends for slip joints (lap joints) for field welding.* Bell ends shall be pressed or rolled to the required shape without hammering. Spigot ends of pipe 30 in. in diameter and larger shall be shaped in or over a die. The longitudinal welds on the inside of the bell end and on the outside of the spigot end on each section of pipe shall be ground to the plate surface. The inside edge of the bell and the outside edge of the spigot end shall be scarfed or lightly ground sufficiently to remove the sharp edges or burrs in the end of the pipe sections.

3.11.2. *Plain or beveled ends for field butt welding.* Ends of pipe sections required for field butt welds either with or without backing-up strips or chill rings shall be plain or beveled.

3.11.3. *Plain ends fitted with butt straps for field welding.* Plain ends of pipe sections fitted with butt straps for field joints shall comply with the details shown on the drawings. Butt straps shall be of the same thicknesses as the corresponding pipe section, shall be of the required width, shall be rolled to fit the outside diameter, and shall be welded to the pipe section. One half of the butt strap shall be welded to the upper half of the end of the pipe section and the other half shall be welded to the lower half of the other end of the pipe section, or a complete butt strap may be fully welded to one end of each pipe section which is to be joined in the field by welding to an adjoining section, or butt straps can be furnished loose, if required. Longitudinal or spiral welds on the outside of the plain end shall be ground to

plate or sheet surface a sufficient distance from the end. The inside edge of the butt strap and the outside edge of the adjacent pipe length shall be ground or sufficiently scarfed to remove sharp edges or burrs.

3.11.4. *Ends for mechanically coupled field joints.* Ends for mechanically coupled field joints shall be as specified by the purchaser and shall be plain, grooved, or banded. The outside ends of plain-end pipe shall be free from surface defects and shall have the longitudinal or spiral welds ground to surface for a sufficient distance from the ends to permit the pipe to make a watertight joint with the coupling. Grooved or banded ends shall be prepared to fit the type of mechanical coupling to be used.

3.11.5. *Ends for riveted field joints.* Ends for riveted field joints shall be as specified by the purchaser and shall be prepared for bumped joints, butt-strap construction, or lap joints, the latter formed by using tapered pipe sections or inside and outside pipe sections. Edges at ends of pipe shall be suitably planed or bevel sheared for field calking. All rivet holes shall be accurately spaced and shall be drilled or subpunched and reamed $\frac{1}{16}$ in. larger than the nominal size of the

rivet. Holes shall be free from burrs and torn or damaged edges.

3.11.6. *Bell-and-spigot ends with rubber gasket.* Bell ends shall be pressed, rolled, or fabricated from plate, sheets, or special sections to the required shape without hammering. Spigot ends shall be formed or fabricated to the required dimensions. The form of the end to retain the gasket shall be in conformance with approved drawings. Longitudinal or spiral welds on the inside of the bell end and the outside of the spigot end shall be ground to the plate surface for a distance not less than the depth of insertion of spigot end into bell end. The gasket shall conform to gasket material requirements set forth in AWWA C300. It shall be shaped for watertightness and shall be furnished by the pipe manufacturer.

3.11.7. *Plain ends fitted with flanges.* Ends to be fitted with flanges shall have the longitudinal or spiral welds on the pipe ground to plate or sheet surface for a distance from the ends to accommodate the flange.

3.11.8. *Tapered ends for driven field joints.* Ends for driven field joints shall be prepared to produce a tight joint.

Section 4—Tolerances

Sec. 4.1—Ends

The ends of each section of pipe shall be accurately measured with a steel tape and shall not exceed the tolerances set forth below, provided, however, that such tolerances do not interfere with the making of the joints.

4.1.1. For a distance of not less than 10 in. from the ends, the outside cir-

cumference of pipe with the following types of ends shall be not less than $\frac{1}{16}$ in. smaller nor more than $\frac{1}{8}$ in. greater than the outside circumference computed from the nominal outside diameter or the nominal inside diameter plus twice the nominal thickness:

a. Plain ends of slip joint (lap joint) pipe for field welding

b. Plain ends with butt straps for field welding

c. Beveled ends fitted with butt straps for field welding

d. Both ends of inside section of inside and outside pipe sections or the smaller end of tapered section for lap riveted field joints

e. Plain ends fitted with flanges.

4.1.2. Ends for mechanical couplings shall have tolerances within the limits required by the manufacturer of the type of coupling to be employed.

4.1.3. For a distance of not less than 10 in. back from the ends the inside circumference of pipe with the following types of ends shall be not less than $\frac{1}{16}$ in. smaller nor more than $\frac{1}{8}$ in. greater than the inside circumference computed from the nominal inside diameter or from the nominal outside diameter less twice the nominal thickness.

a. Ends plain or beveled for field butt welding

b. Bumped ends for riveted field joints

c. Ends for riveted field joints.

The tolerance for tapered pipe shall apply at the smaller end only.

4.1.4. The inside diameter of the bell of slip joint pipe for field welding shall be $\frac{1}{32}$ — $\frac{3}{16}$ in. greater than the outside diameter of the plain end. The inside diameter of the outside section of inside and outside pipe sections or the inside diameter of the larger end of tapered sections for lap riveted field joints shall be $\frac{1}{16}$ — $\frac{1}{8}$ in. greater than the outside diameter of the inside section or smaller end of the tapered section; such outside diameter shall be the nominal outside diameter or shall be equal to the nominal inside diameter plus twice the nominal thickness.

4.1.5. The out-of-roundness of pipe ends at the fabricator's plant when measured on any diameter shall be within limits consistent with the type of field joint specified and the type of corrosion protection to be employed.

4.1.6. The ends of all pipe sections shall not vary more than $\frac{1}{8}$ in. at any point from a true plane at right angles to the axis of the pipe.

Sec. 4.2—Circumference

The outside circumference of pipe 20 in. in diameter and larger shall not vary more than 0.5 per cent from the size specified, except that the circumference at ends shall be sized, if necessary, to meet the requirements of this standard. The outside circumference of pipe less than 20 in. in diameter shall not vary more than 1 per cent from the size specified.

Sec. 4.3—Straightness

Finished pipe sections shall be truly straight with walls parallel to the axis of the pipe. Any pipe section with a fault in alignment exceeding $\frac{1}{8}$ in. for each 10 ft in length from a line parallel to the axis of the pipe will be rejected.

Sec. 4.4—Length

Straight pipe sections shall not vary from the specified length by more than 2 in. Special sections shall not vary from the specified length by more than $\frac{1}{8}$ in.

Sec. 4.5—Weld Bead

The height of the outside weld bead above the contour of the plate surface shall be not more than $\frac{1}{8}$ in. Beads exceeding $\frac{1}{8}$ in. in height shall be removed by grinding or chipping. All longitudinal, spiral, and girth welds

on the inside of the pipe shall be milled, ground, or chipped down so that the height of weld bead shall be not more than $\frac{1}{16}$ in. above the contour of the plate surface. Milling, grinding, or chipping of the weld bead below the surface of the plate shall not be permitted.

Sec. 4.6—Bell-and-Spigot Ends With Rubber Gasket

The difference in circumferential measurement between the outside circumference of the spigot and the inside circumference of the bell shall be no less than 0.060 in. and no more than 0.200 in.

Section 5—Tests

Sec. 5.1—Weld Test Specimens

5.1.1. Weld test specimens shall be furnished the inspector approximately every 300 ft of pipe and shall be furnished more often if, in the judgment

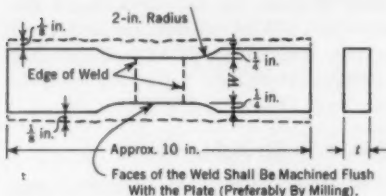


Fig. 1. Reduced-Section Tensile-Test Specimen

If coupons have been cut apart by a fusion process, the flame-cut surfaces are to be machined off as indicated. The width (W) of the machined specimen shall be approximately 1 1/2 in. if t is equal to or less than 1 in.; if t is greater than 1 in., W shall be approximately 1 in.

of the inspector, a satisfactory weld is not being made. Weld specimens may be cut from the pipe along the longitudinal seam, spiral seam, circumferential seam or from test plates welded to ends of pipe sections as may be required by the inspector. The preparation of the edges of the test plates and the welding processes, procedures, and technique shall be the same as those used in the fabrication of the pipe.

5.1.2. Tests of the specimen plates shall be made at the purchaser's expense, and such tests shall be performed at the purchaser's laboratory, at a commercial laboratory designated by the purchaser, or at the contractor's plant, if facilities are available. The contractor will be permitted to insert a patch in the pipe. Where feasible, the patch shall be welded, both inside and outside, filling the area from which the specimens are cut, except that, if specimens are cut from the end of the pipe, the ends of the pipe shall be cut off to the extent necessary.

5.1.3. The test specimen plates shall be of sufficient size to obtain the required test specimens.

5.1.4. Two etch or waste specimens, one reduced-section tensile-test specimen, one free-bend test specimen, one reverse-bend test specimen, and one nick break test specimen shall be taken from each test specimen plate. A $\frac{3}{8}$ -in. clearance shall be left between specimens to permit milling and machining of specimens to size. Flame-cut edges of test specimens shall be prepared as follows:

a. Etch specimens. Each specimen shall be $\frac{1}{2}$ in. wide and shall be obtained from each end of the test specimen plate. The two cross-sectional surfaces of the welded joint specimen

shall be polished to bright, smooth surfaces. This shall be accomplished by filing and polishing with emery cloth and shall be completed with the use of emery cloth of 00 grade. If the specimens are removed by flame cutting they shall be machined, ground, or filed to a smooth surface by the removal of not less than $\frac{1}{8}$ in. of material below the flame-cut surface and they shall then be polished as above. The specimens shall then be etched in a

shape and dimensions shown in Fig. 2. The free-bend test specimens shall be bent transversely to the welded joint with the wider portion of the weld surface on the outside of the bend. The bend in the specimen may be started by holding the specimen in a vise about one-third from the end, producing an initial bend at this point with hammer blows. Another bend about one-third from the other end may be produced in the same manner. The specimen with

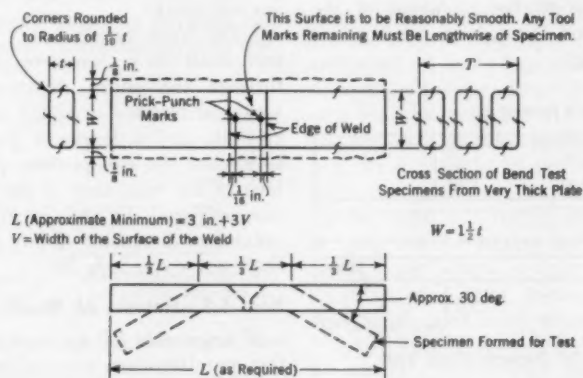


Fig. 2. Free-Bend Test

If coupons have been cut apart by a fusion process, the flame-cut surfaces are to be machined off as indicated. The symbol "f" indicates a light finish cut; weld reinforcement is to be removed. The length (L) of the bend specimen is immaterial provided the bend occurs at the weld.

boiling solution of equal parts of hydrochloric acid and water for a sufficient period of time to reveal any lack of soundness that might exist at the cross-sectional surfaces of the specimen.

b. Reduced-section tensile-test specimens. Reduced-section tensile-test specimens shall be of the size and dimensions shown in Fig. 1.

c. Free-bend test specimens. Free-bend test specimens shall be of the

the initial bend at each end and at atmospheric temperature is then placed as a strut in a vice or compression machine and the pressure applied gradually—that is, without shock—at the ends until failure occurs in the outside fibers of the bend specimen. When a crack is observed in the convex surface of the specimen between the edges, the specimen shall be considered to have failed and the test shall

be stopped. Cracks at the corners of the specimen shall not be considered as a failure. The appearance of small defects in the convex surface shall not be considered as a failure if their greatest dimension does not exceed $\frac{1}{16}$ in.

d. Reverse-bend test specimens. Reverse-bend test specimens shall be of the shape and dimensions as shown in Fig. 3. The reverse-bend test specimen for single-welded butt joints without a backing-up strip shall be bent around a pin or mandrel whose diameter is equal to the thickness of the specimen and through an angle of at least 30 deg.

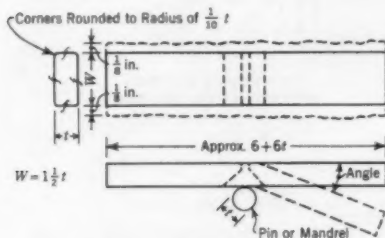


Fig. 3. Reverse-Bend Test

If coupons have been cut apart by a fusion process, the flame-cut surfaces are to be machined off as indicated. The symbol "f" indicates a light finish cut; weld reinforcement is to be removed.

e. Nick break test specimen. The nick break test specimen shall be of the shape and dimensions shown in Fig. 4. It shall be suitably supported and broken in the center of the weld by a sharp blow or blows of a hammer.

Sec. 5.2—Results of Weld Tests

The minimum requirements for the test results are given below:

5.2.1. Reduced-section tensile test. Yield point and ultimate tensile strength shall be not less than the mini-

um of the specified tensile range of the plate used.

5.2.2. Free-bend test. The elongation shall be measured within or across approximately the entire weld on the outside of the fibers of the test specimen and shall be at least 20 per cent. The widths of any surface crack shall be deducted.

5.2.3. Reverse-bend test. There shall be no defects due to lack of fusion between weld metal and plate and there shall be no cracks other than small surface cracks.

5.2.4. Nick break test. The fracture shall show complete penetration through the entire thickness of the weld, an absence of oxide or slag inclusions, and a degree of porosity not to exceed six gas pockets per square inch of the total area of the weld surface exposed in the fracture. The maximum dimension of any gas pocket shall not exceed $\frac{1}{16}$ in.

Sec. 5.3—Retests of Welds

If any welds fail to meet any of the test requirements, two additional test specimen plates shall be cut from the pipe section from which the original specimen plate was obtained, and retests shall be made on these additional specimens. Should any of the tests made from the second test specimen plates fail to meet the requirements, all pipe sections in the lot represented by the pipe section whose retest failed, as determined by the inspector, will be rejected. All pipe sections in this lot, other than the pipe section from which the test specimens were obtained, may be reconditioned and re-submitted for inspection. The inspector may require as many specimen plates as he desires from this lot and all tests shall meet the requirements, or all pipe sections in this lot

will be rejected. The inspector may require that the welding operator whose retest failed shall not be permitted to perform any further welding required under this standard. The contractor will be charged by the purchaser for all expenses incurred in making retests.

Sec. 5.4—Hydrostatic Tests

5.4.1. *Pipe 30 in. in diameter and larger.* Each straight pipe section and tapered section whose ends are in planes at right angles to the pipe axis shall be tested hydrostatically in a test-

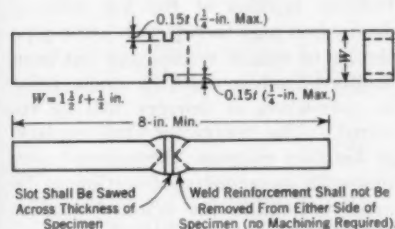


Fig. 4. Standard Bar for Nick Break Test

The specimen shall be suitably supported and broken in the center by a sharp blow or blows of a hammer.

ing machine. The test pressure shall be that specified by the purchaser, but shall not exceed that determined by the following formula:

$$P = \frac{2st_1}{D}$$

in which P is the test pressure (in pounds per square inch); s is the allowable fiber stress (in pounds per square inch), which shall be equal to 85 per cent of the yield point of the specified plate; t is the wall thickness (in inches) required for the specified working water pressure; and D is the

inside diameter (in inches) of straight pipe sections or the larger inside diameter of tapered sections.

5.4.2. *Pipe less than 30 in. in diameter.* All straight pipe less than 30 in. in diameter shall be tested at the shop to at least the hydrostatic pressures specified in Table 2 of the Appendix to this standard, and the pressures shall be maintained for not less than 5 sec.

5.4.3. *Fittings.* Fittings made in accordance with the requirements of this standard by certified welding operators from tested pipe shall not be subjected to a hydrostatic test. All fittings not fabricated from tested pipe shall be subjected to a hydrostatic test pressure specified by the purchaser but not exceeding $1\frac{1}{2}$ times the working water pressure. Sections that cannot be tested in a testing machine may be prepared for testing by welding on heads, or as otherwise approved by the engineer, and after testing the ends shall be reconditioned when necessary.

5.4.4. *Test procedure.* Vents shall be provided at all high points of the pipe to prevent the formation of air pockets while the pipe is being filled, and before the application of pressure the equipment shall be inspected to see that it is tight. The required pressure shall be maintained generally for 2 min and occasionally for 5 min. While under pressure the welded seams shall be inspected for leaks and each leak shall be marked plainly. In this test no hammering shall be permitted. Any section showing more than one leak for each 5 ft of welded seam, or more than 2.5 per cent of the total length of welded seam, will be rejected. Closing leaks by means of a calking tool shall not be permitted. In sections showing less than one leak for each 5 ft of welded seam and not

more than 2.5 per cent of the total length of the welded seam, the leaks shall be repaired by chipping the weld and rewelding by hand, after which the pipe shall be retested hydrostatically. If, on retest, a pipe section shows any leaks in the welded seams it shall be

rejected. Accepted pipe sections shall be stamped after test by the inspector with some legible mark of identification. In place of the inspector's stamp, the purchaser may accept a certificate from the contractor concerning these tests.

Section 6—Marking and Delivery of Pipe

Sec. 6.1—Marking

The date of manufacture and a serial number for identification shall be conspicuously painted in white on the inside of each section of pipe and each special section. If the pipe is coated such marking shall be done at the shop and later transferred to the inside coating. The contractor may be required to furnish the engineer with field sketches showing where each numbered pipe or special section goes in the line, and the numbers on such sections shall correspond with those painted on the pipes and special sections.

Sec. 6.2—Delivery

Pipe sections and special sections shall be delivered in the order required by the purchaser. The transporting and handling of coated or lined pipe shall be in accordance with the purchaser's supplementary specifications.

Sec. 6.3—Finished Pipe at Delivery Destination

6.3.1. Any pipe section or special section that shows dents, kinks, abrupt changes in curvature other than specified, or injuries at the fob delivery destination may be rejected. Any pipe section or special section that has been dropped from a truck or crane prior to completion of delivery will be rejected. The contractor shall replace, at his own expense, the rejected section with an undamaged section or he may recondition the rejected section.

6.3.2. Rejected sections shall be reconditioned by rerolling or by pressure but not by hammering and shall be retested hydrostatically to the required pressure if deemed necessary by the inspector. All sections dropped from truck or crane shall be reconditioned as stated above and shall be retested hydrostatically to the required pressure.

Appendix

Sec. A1—Intermediate-Strength Steel Plate

In connection with large-diameter steel pipe, where thickness is governed by internal pressure and high water

pressures exist, consideration should be given to using a higher-strength steel than those specified in Sec. 2.1.1. This may result in substantial saving in cost of the steel pipe without sacri-

TABLE 1

Dimensions, Weights, and Test Pressures for Fabricated Steel Pipe, 4-28 in. OD*

Nom. Size, OD in.	Wall Thickness in.	Weight lb./ft	Test Pressure psi	Nom. Size, OD in.	Wall Thickness in.	Weight lb./ft	Test Pressure psi	
4	0.105	4.3	1,000	16	0.135	22.8	300	
	0.135	5.5	1,200		0.179	30.2	400	
4½	0.105	4.9	900		0.188	31.6	400	
	0.135	6.2	1,000		0.239	40.2	550	
6	0.105	6.6	600		0.250	42.0	550	
		8.4	800	0.312	52.3	700		
		11.6	1,100	18	0.135	25.7	250	
		13.5	1,300		0.179	34.0	350	
	6½	0.105	7.3		550	0.188	35.6	350
0.135		9.3	700		0.239	45.3	450	
0.188		12.8	1,000		0.250	47.3	500	
0.219		14.9	1,200	0.312	59.0	600		
8	0.105	8.8	450	20	0.135	28.6	250	
		11.3	600		0.179	37.8	300	
		14.9	800		0.188	39.6	350	
		0.188	15.6		850	0.239	50.4	400
	0.239	19.8	1,100		0.250	52.7	450	
8½	0.105	9.5	400	22	0.312	65.7	550	
		0.135	12.2		550	0.375	78.5	700
		0.179	16.1		750	0.179	41.7	300
		0.188	16.8		750	0.188	43.6	300
	0.239	21.4	1,000		0.239	55.5	400	
10	0.135	14.2	450	24	0.250	58.0	400	
		18.7	650		0.312	72.3	500	
		19.6	700		0.375	86.6	600	
		0.239	24.9		850	0.179	45.5	250
	10½	0.135	15.3		400	26	0.188	47.6
0.179			20.2	600	0.239		60.6	350
0.188			21.1	650	0.250		63.4	350
0.239			26.8	800	0.312		79.0	450
12	0.135	17.1	400	28	0.375		94.6	550
		22.5	550		0.438	110.0	650	
		23.6	550		0.500	125.4	750	
		0.239	30.0		700	0.179	49.3	250
	12½	0.135	18.1		350	30	0.188	51.6
24.0			500	0.239	65.7		300	
25.1			500	0.250	68.7		350	
0.239			31.9	650	0.312		85.7	400
14		0.135	19.9	350	32		0.375	102.6
	26.4		450	0.438		119.4	600	
	27.6		450	0.500		136.1	700	
	0.239		35.1	600		0.179	53.1	250
	14½	0.135	20.0	300		34	0.188	55.6
26.0			400	0.239	70.8		300	
27.0			400	0.250	74.0		300	
0.239			33.0	500	0.312		92.4	400
15		0.135	20.5	300	36		0.375	110.6
	26.5		400	0.438		128.7	550	
	27.5		400	0.500		146.8	650	
	0.239		33.5	500				

* Other diameters and wall thicknesses can be furnished by some manufacturers.

ficing quality or security. For this purpose, the following specification is recommended:

Steel plate shall comply with the chemical and physical requirements of API Specifications for High Test Line Pipe, Standard 5LX, Grade X42, and shall conform to the applicable requirements of the latest revision of ASTM A6 (Delivery of Rolled Steel Plates, Shapes,

Sheet Piling, and Bars for Structural Use).

Sec. A2—Physical Data

A2.1. Dimensions, weights, and test pressures for fabricated steel pipe of sizes 4–28 in. (OD) are given in Table 1.

A2.2. Dimensions and weights of fabricated steel pipe of sizes 30–96 in. (ID) are given in Table 2.

TABLE 2
Dimensions and Weights for Fabricated Steel Pipe, 30–96 in. ID*

Nom. Size, ID in.	Wall Thickness—in.									
	0.188	0.250	0.312	0.375	0.438	0.500	0.625	0.750	0.875	1.000
	Weight—lb/ft									
30	62.2	83.1	103.9	124.6	145.4	166.1				
32	66.8	88.9	110.8	132.9	154.9	177.0				
34	71.0	94.4	117.6	141.1	164.5	187.9				
36	75.5	100.1	124.8	149.6	174.5	199.2	249.3			
38	79.9	105.7	131.7	157.9	183.8	210.2	262.9			
40	82.4	110.1	137.5	164.9	192.0	220.0	275.9			
42	86.5	115.6	144.3	173.1	201.7	230.9	289.5			
45		123.8	154.6	185.3	216.0	247.2	309.8			
48		132.3	165.6	198.5	231.4	264.1	330.2	396.3		
51		140.6	175.9	210.9	245.7	280.5	350.6	420.7		
54		149.1	186.2	223.2	260.0	296.8	371.0	446.2		
57		157.4	196.5	235.5	274.4	313.1	391.3	470.6		
60		165.6	206.8	247.8	288.7	329.5	411.7	495.1	578.8	662.8
63		174.3	217.1	260.1	303.0	345.8	432.1	519.5	607.3	695.4
66		182.6	227.4	272.4	317.4	362.2	452.5	544.0	635.8	728.0
69		191.3	238.2	285.4	332.5	379.4	474.0	568.4	664.4	758.8
72		199.6	248.5	297.8	346.9	395.8	494.4	592.9	692.9	791.3
75		207.9	258.8	310.1	361.2	412.2	514.8	617.3	721.4	823.8
78		214.6	267.9	320.9	373.8	426.5	534.0	640.2	748.1	856.3
81		223.4	278.8	334.0	389.0	443.9	554.3	666.2	778.5	891.0
84		231.6	289.1	346.3	403.4	460.2	574.7	690.7	807.0	923.6
87		239.9	299.4	358.6	417.7	476.5	595.1	715.1	835.5	956.2
90		248.1	309.7	371.0	432.0	492.9	615.5	739.6	864.0	988.8
93		257.0	319.9	383.3	446.4	509.2	635.8	764.0	892.6	1,021.0
96		265.3	329.4	395.6	460.7	525.6	656.2	788.5	921.1	1,054.0

* Other diameters and wall thicknesses can be furnished by some manufacturers. Weights are the theoretical weight plus overweight tolerance, which varies from about 1.75 per cent for pipe 1/2 in. thick to a maximum of 4.5 per cent for pipe 3/16 in. thick, or an average of approximately 2.5 per cent.



AWWA C202-60T

American Water Works Association

TENTATIVE
AWWA STANDARD
for
MILL-TYPE STEEL WATER PIPE

This "Standard for Mill-Type Steel Water Pipe" is based upon the best known experience and is intended for use under normal conditions. It is not designed for unqualified use under all conditions, and the advisability of its use for any installation must be subjected to review by the engineer responsible.

First Edition approved Apr. 25, 1940; Second Edition approved as "Tentative" May 15, 1960.

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Acknowledgment is made and appreciation is expressed to the Subcommittee on Pipe of the Steel Water Pipe Manufacturers Technical Advisory Committee for its assistance in the preparation of this standard. The officers of this committee are R. C. Beam, chairman; Walter H. Cates, secretary.

Latest Revisions to C202

This Second Edition of AWWA Standard C202 was approved as "Tentative" on May 15, 1960. The principal change involved is that this edition covers mill-type pipe in all sizes, whereas the First Edition covered both fabricated and mill-type pipe in sizes up to, but not including, 30 in.

Designation. As a result of the revision, the former designation, "C202-59," has been changed to "C202-60T."

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AWWA C202-60T

Tentative AWWA Standard for Mill-Type Steel Water Pipe

Section 1—General

Sec. 1.1—Scope

This standard covers mill-type steel pipe intended for the conveyance of water. It covers pipe in all sizes; in single or double random lengths or specified laying lengths; in straight sections or other special sections required for closures, curves, or changes in alignment or grade to meet special conditions; and with ends prepared for the type of joint specified. For the purpose of this standard, mill-type pipe is defined as furnace-welded, electrically welded, or seamless steel pipe of any size produced to meet finished-pipe specifications. The types of pipe covered by this standard shall be limited by the following provisions:

1.1.1. *Furnace-welded pipe.* Furnace-welded pipe shall be continuous butt welded or furnace butt welded.

1.1.2. *Electrically welded pipe.* Electrically welded pipe shall be of Grade A, B, or X42,* as specified by the purchaser, and shall be induction butt welded; spiral- or straight-seam

resistance butt welded; or spiral- or straight-seam fusion butt welded. Fusion-butt-welded pipe shall be manufactured by automatic submerged-arc or automatic shielded-arc welding.

1.1.3. *Seamless pipe.* Seamless pipe shall be of Grade A, B, or X42, as specified by the purchaser.

Sec. 1.2—Definitions

Under this standard, the following definitions shall apply:

1.2.1. *Purchaser:* The person, firm, corporation, or governmental subdivision entering into a contract or agreement for the purchase of any materials or work to be performed under this standard.

1.2.2. *Contractor:* The person, firm, or corporation executing the contract or agreement with the purchaser to furnish any material or perform any work.

1.2.3. *Inspector:* The duly authorized representative of the purchaser qualified by training and experience in the manufacture of steel pipe of the character covered by this standard.

1.2.4. *Bevel:* An angle applied on the end of a pipe and measured from a line perpendicular to the axis.

1.2.5. *Butt weld:* A weld whose throat lies in a plane disposed approximately 90 deg with respect to the surfaces of at least one of the parts joined.

* Grades A and B under this standard are similar to grades A and B for pipe under American Petroleum Institute API 5L (Specifications for Line Pipe). Grade X42 is similar to Grade X42 for pipe under API 5LX (Specifications for High-Test Line Pipe). The purchaser need not make reference to the API specifications in using this standard, however.

The size of a butt weld is expressed in terms of its net or unreinforced throat dimension in inches. A double-welded butt joint is one in which the filler metal is added to both sides. A single-welded butt joint is one in which the filler metal is added to one side only.

1.2.6. Fillet weld: A weld of approximately triangular cross section whose throat lies in a plane disposed approximately 45 deg with respect to the surfaces of the parts joined. The size of the fillet weld is expressed in terms of the width, in inches, of one of its adjacent fused legs, the shorter if unequal.

1.2.7. Flame cutting: The process of severing metal by means of a gas flame.

1.2.8. Furnace-welded pipe: Either bell-welded or continuously welded pipe.

1.2.9. Bell-welded pipe: Pipe produced in individual lengths from cut-length skelp, having its longitudinal butt joint forge welded by the mechanical pressure developed in drawing the furnace-heated skelp through a cone-shaped die (commonly known as a "welding bell") which serves as a combined forming and welding die.

1.2.10. Continuous-welded pipe: Pipe produced in continuous lengths from coiled skelp and subsequently cut into individual lengths, having its longitudinal butt joint forge welded by the mechanical pressure developed in rolling the hot-formed skelp through a set of round-pass welding rolls.

1.2.11. Fusion welding: The process of welding metals in the molten or molten and vaporous state, without the application of mechanical pressure or blows.

1.2.12. Girth weld: A circumferential welded seam lying in one plane,

used to join sections into lengths of straight pipe or to join pieces of mitered pipe to form fabricated special sections and fittings.

1.2.13. Grade or class of material: The designation of material as to chemical analysis or physical properties.

1.2.14. Ladle analysis: The chemical analysis taken at the time the steel is cast into ingots.

1.2.15. Manufacturing process: The designation of pipe in accordance with the method of manufacture (see Sec. 1.1).

1.2.16. Mill-type pipe: Seamless, furnace-welded, electrically welded, or furnace-butt-welded steel pipe of any size produced to meet finished-pipe specifications.

1.2.17. Nominal outside diameter: The commercial name or given dimension for pipe in sizes up to 12 in., as distinguished from the actual or measured dimension.

1.2.18. Nominal size: The commercial name or given dimension for pipe in sizes up to 12 in., as distinguished from the actual or measured dimension.

1.2.19. Nominal wall thickness: The named or given thickness as distinguished from the actual or measured thickness.

1.2.20. Nominal weight per foot (for uncoated and unlined pipe): The theoretical weight per foot, as shown in published tables, as distinguished from the actual or measured weight per foot of the finished pipe.

1.2.21. Plain-end pipe: Pipe cut off to random or exact lengths, but not threaded. It is not beveled unless so specified.

1.2.22. Resistance-butt-welded pipe: Pipe having a longitudinal or spiral butt joint in which coalescence is produced by the heat obtained from re-

sistance of the pipe to the flow of electric current in a circuit of which the pipe is a part, and by the application of pressure.

1.2.23. *Root*: The zone at the bottom of the cross-sectional space provided to contain the deposited filler metal.

1.2.24. *Seamless pipe*: Pipe without welds made from solid ingots, blooms, billets, or round bars which have been hot pierced and then brought to the desired size by hot rolling, hot drawing, or a combination of both.

1.2.25. *Specified lengths*: Sections of finished pipe whose length dimensions do not vary from a fixed figure specified by the purchaser by more than the tolerance set forth in this standard.

1.2.26. *Spiral-seam fusion-welded pipe*: Pipe in which the line of the fusion seam forms a helix on the barrel of the pipe.

1.2.27. *Straight-seam fusion-welded pipe*: Pipe in which the line of the fusion seam parallels the axis of the pipe.

1.2.28. *Throat*: The thickness of deposited weld metal, not including reinforcing, along a straight line passing through the root.

1.2.29. *Handling-tight coupling*: A threaded coupling sufficiently tight that a wrench is necessary to loosen it.

1.2.30. *Power-tight coupling*: A threaded coupling sufficiently tight that it cannot be loosened by the application of a manual wrench alone.

Sec. 1.3—Supplementary Details to Be Specified by Purchaser

When purchasing pipe under this standard, the purchaser shall provide specific supplementary information in

this specifications regarding the following details:

1.3.1. Outside diameter.

1.3.2. Wall thickness.

1.3.3. Grade or class designation.

1.3.4. Minimum mill hydrostatic test pressure.

1.3.5. Type of field joints (description or drawings).

1.3.6. Lengths—that is, whether single random (average length 17½ ft), double random (average length 35 ft), or specified cut length (see Sec. 7.4.1 for percentage of minimum and maximum lengths).

1.3.7. The maximum number of longitudinal and girth seams permitted in each length.

1.3.8. All special sections and special fittings, indicating, for each component part, the dimensions and type of field joint together with length tolerances.

1.3.9. Whether a certified statement from the contractor that the required inspection and tests have been made is desired.

1.3.10. Any exceptions to variations from this standard for special requirements for further processing.

Sec. 1.4—Inspection

1.4.1. All work done and material furnished under this standard may be inspected by the purchaser, but such inspection shall not relieve the manufacturer of his responsibility to furnish material and perform work in accordance with this standard. If the inspector desires to inspect the pipe or witness the tests, reasonable notice shall be given of the time at which the inspection is to be made. When requested by the purchaser, a certified statement from the manufacturer that the required inspection and tests have been made shall be furnished.

1.4.2. At all times while work on the contract is being performed the inspector shall have free entry to all parts of the manufacturer's works which concern the manufacture of the pipe ordered. The manufacturer shall afford the inspector, without charge, all reasonable facilities to satisfy him that the pipe is being manufactured in accordance with this standard. All inspections shall be made at the place of manufacture prior to shipment and shall be so conducted as not to interfere unnecessarily with the operation of the works.

1.4.3. The inspector may reject any or all pipe sections or special sections that do not conform to the prescribed test results, conditions of pipe, and tolerances set forth in this standard, the approved drawings, and in the purchaser's supplementary specifications.

Sec. 1.5—Workmanship

1.5.1. *Defects.* The finished pipe shall be free from injurious defects. Defects in seamless pipe or in the parent metal of welded pipe shall be considered injurious when the depth of defect is greater than 12.5 per cent of the tabulated wall thickness. All cracks, sweats, and leaks in welds shall be considered injurious. All end weld-

ing of longitudinal seams of fusion-welded pipe, if not done by automatic submerged-arc or automatic shielded-arc welding, shall be done by a procedure and a welder qualified in accordance with Sec. 6.

1.5.2. *Repair of defects.* The repair of injurious defects shall be permitted at the discretion of the manufacturer, except that the repair of defects in seamless pipe or in the parent metal of welded pipe shall not be permitted if the depth of defect exceeds one-third of the tabulated wall thickness of the pipe and the length of that portion of the defect in which the depth exceeds 12.5 per cent is greater than 25 per cent of the tabulated outside diameter of the pipe, or if more than one repair is required in any length equivalent to ten times the tabulated outside diameter of the pipe. Repairs shall conform to the following requirements:

a. The defect shall be completely removed and the cavity thoroughly cleaned.

b. The repair weld shall be made by either automatic or manual welding, by a procedure and a welder qualified in accordance with Sec. 6.

c. Each length of repaired pipe shall be tested hydrostatically in accordance with Sec. 4.1.

Section 2—Chemical Properties and Tests

Sec. 2.1—Chemical Properties

The steel furnished for pipe under this standard shall be of good welding quality and shall conform to the chemical requirements shown in Table 1 for the grade of pipe ordered. Steel may be made by any of the following processes, in accordance with the method of pipe manufacture: open hearth, elec-

tric furnace, basic oxygen, killed de-oxidized acid, or basic bessemer.

Sec. 2.2—Tests

2.2.1. *Ladle analysis.* When requested by the purchaser, the manufacturer shall furnish a report giving the ladle analysis of each heat of open-hearth, electric-furnace, or basic-oxygen

steel and such analyses as the manufacturer may customarily make of bessemer steel used in the manufacture of pipe conforming to this standard. The analyses so determined shall conform to the requirements specified in Table 1.

2.2.2. Check analysis. When so specified on the purchase order, the manufacturer shall furnish a report of check analysis on finished pipe 2 $\frac{3}{8}$ in. and larger. The number of samples for check analysis shall be one from each of two lengths of pipe from each lot of 400 lengths or fewer of each size 2 $\frac{3}{8}$ in. through 5 $\frac{1}{8}$ in., one from each of two lengths from each lot of 200

TABLE 1

Chemical Requirements for Ladle Analysis

Type of Steel	Maximum Content—per cent		
	Manganese	Phosphorus	Sulfur
Furnace butt welded	0.65	0.11	0.06
Grade A	1.25	0.04	0.05
Grade B	1.25	0.11	0.05
Grade X42	1.25	0.10	0.05

lengths or fewer of each size 6 $\frac{5}{8}$ in. through 20 in., and one from each of two lengths from each lot of 100 lengths or fewer of each size 22 in. and larger. For multiple-length pipe, a length shall be considered as all of the sections cut from a particular multiple length. The samples shall be taken as follows:

a. Seamless pipe. At the option of the manufacturer, cuttings or drillings for check analysis shall be taken either from tensile test specimens or from the pipe at several points around the finished pipe. If drillings are used, the minimum drill size shall be $\frac{1}{8}$ in. and drillings shall be taken by drilling all the way through the pipe wall.

b. Welded pipe. At the option of the manufacturer, cuttings or drillings for check analysis shall be taken from the finished pipe, plate, skelp, tensile-test specimens, or flattening-test specimens. The location of the samples shall be approximately 90 deg from the weld. Samples shall be taken in such a manner as to represent the total wall thickness. The minimum drill size shall be $\frac{1}{8}$ in. The composition so determined shall conform to the ladle analysis limits, and permissible variations shall not exceed the following amounts: manganese, +0.05 per cent; phosphorus, +0.01 per cent; sulfur, +0.01 per cent.

2.2.3. Recheck analysis. If the check analysis of both lengths of pipe representing the lot fails to conform to the specified requirements, at the manufacturer's option, either the lot shall stand rejected or all the remaining lengths in the lot shall be tested individually for conformance to the specified requirements. If only one of the two samples fails, at the manufacturer's option, either the lot shall stand rejected or two recheck analyses shall be made on two additional lengths from the same lot. If both recheck analyses conform to the requirements, the lot shall be accepted except for the length represented by the initial analysis which failed. If one or both of the recheck analyses fail, at the manufacturer's option, the entire lot shall be rejected or each of the remaining lengths shall be tested individually. In the individual testing of the remaining lengths in any lot, analysis for only the disqualifying element or elements need be determined. Samples for recheck analysis shall be taken in the same manner as specified for check analysis samples.

Section 3—Mechanical Properties and Tests

Sec. 3.1—Mechanical Properties

Pipe furnished under this standard shall conform to the tensile properties specified in Table 2 for the particular grade ordered. The yield strength shall be the tensile stress required to produce a total elongation, under load, of 0.5 per cent of the gage length, as determined by an extensometer or by multiplying dividers. Minimum elongation requirements for seamless and electrically welded pipe shall be those in Table 3 for the particular grade ordered. If the tabulated wall thickness of seamless or electrically welded pipe is other than those listed in Table 3 or less than 0.312 in., the minimum elongation requirement shall be determined by one of the following formulas:

Grade	Formula
A	$E = 56t + 17.50$
B	$E = 48t + 15.00$
X42	$E = 40t + 12.50$

in which E is the minimum elongation in 2 in. (in per cent) and t is the tabulated wall thickness (in inches). Minimum elongation and gage length requirements for furnace-butt-welded pipe 1.050 in OD and smaller shall be those set forth in Table 4.

TABLE 2

Minimum Tensile Requirements for Mill-Type Steel Water Pipe

Type of Steel	Tensile Strength psi	Yield Point psi
Furnace butt welded	45,000	25,000
A	48,000	30,000
B	60,000	35,000
X42	60,000	42,000

TABLE 3

Minimum Elongation Requirements for Seamless and Electrically Welded Mill-Type Steel Water Pipe

Tabulated Wall Thickness in.	Grade A	Grade B	Grade X42
	Elongation in 2 in.—per cent		
0.312 and larger	35.00	30.00	25.00
0.281	33.25	28.50	23.75
0.250	31.50	27.00	22.50
0.219	29.75	25.50	21.25
0.188	28.00	24.00	20.00
0.156	26.25	22.50	
0.124	24.50	21.00	
0.094	22.75	19.50	
0.062	21.00	18.00	

Sec. 3.2—Tensile Tests

For seamless and furnace-butt-welded pipe in all sizes and for electric-welded pipe 6 $\frac{5}{8}$ in. and smaller, the tensile properties shall be determined by tests on longitudinal specimens conforming to the requirements of Sec. 3.2.1. For electric-welded pipe 8 $\frac{5}{8}$ in. and larger, the tensile-test specimens may be taken transversely; specimens shall conform to the requirements of Sec. 3.2.2. Tensile tests on all longitudinal specimens and on all transverse body test specimens shall include yield strength, tensile strength, and elongation determinations. At the option of the manufacturer, transverse yield strength may be determined by the ring expansion method. Transverse weld test specimens need be tested for tensile strength only. All tensile tests shall be made with the specimens at room temperature.

3.2.1. *Longitudinal tensile-test specimens.* Longitudinal tensile-test speci-

mens shall be either full-section specimens or strip specimens, at the option of the manufacturer. Strip specimens from seamless pipe may be taken from any location, at the option of the manufacturer. Strip specimens shall be taken approximately 90 deg from the weld or, at the option of the manufacturer, from the skelp, parallel to the direction of rolling and approximately midway between the edge and the center. All longitudinal strip specimens shall be approximately $1\frac{1}{2}$ in. wide in the gage length if suitable curved-face testing grips are at hand; otherwise, they shall be approximately $\frac{3}{4}$ in. wide for pipe $3\frac{1}{2}$ in. and smaller, approximately 1 in. wide for pipe 4 in. through 6 $\frac{3}{8}$ in., and approximately $1\frac{1}{2}$ in. wide for pipe 8 $\frac{3}{8}$ in. and larger. Longitudinal test specimens shall represent the full wall thickness of the pipe from which the specimens were cut and shall be tested without flattening.

3.2.2. Transverse tensile-test specimens. Transverse body test specimens from electric-welded pipe shall be taken opposite the weld; transverse weld test specimens shall be taken with the weld at the center of the specimen. All transverse test specimens shall be approximately $1\frac{1}{2}$ in. wide in the gage length, and shall represent the full wall thickness of the pipe from which the specimens were cut.

3.2.3. Number of tensile tests required. One tensile test shall be made on one length of pipe from each lot of 400 lengths or fewer of each size 6 in. and smaller, from each lot of 200 lengths or fewer of each size 6 $\frac{3}{8}$ in. through 20 in. and from each lot of 100 lengths or fewer of each size 22 in. and larger. For multiple-length pipe, a length shall be considered as all of the sections cut from a particular mul-

tip length. For electric-welded pipe, one transverse tensile test shall be made of the weld on a length of pipe from each lot of 200 lengths or fewer of each size 8 $\frac{3}{8}$ in. through 12 $\frac{3}{8}$ in. and from each lot of 100 lengths or fewer of each size 14 in. and larger.

3.2.4. Retest. If the tensile-test specimen representing a lot of pipe fails to conform to the specified requirements, the manufacturer may elect to make retests on two additional lengths from the same lot. If both of the retest specimens conform to the requirements, all the lengths in the lot shall be accepted except the

TABLE 4

Minimum Elongation and Gage Length Requirements for Furnace-Butt-Welded Pipe 1.050 in. OD and Smaller

Outside Diameter in.	Nominal Size in.	Gage Length in.	Elongation in 8 in. per cent
1.050	$\frac{3}{4}$	6	18
0.840	$\frac{1}{2}$	6	18
0.675	$\frac{3}{8}$	4	18
0.540	$\frac{1}{4}$	4	18
0.405	$\frac{1}{8}$	2	18

length from which the initial specimen was taken. If one or both of the retest specimens fail to conform to the specified requirements, the manufacturer may elect to test individually the remaining lengths in the lot, in which case, determinations are required only for the particular requirements with which the specimens failed to comply in the preceding tests. Specimens for retests shall be taken in the same manner as specified in Sec. 3.2.1 and Sec. 3.2.2. If any tensile-test specimen shows defective machining or develops flaws, it may be discarded and another specimen substituted. When the elon-

gation of any tensile-test specimen is less than that specified, if any part of the fracture is outside of the middle third of the gage length as indicated by scribe scratches marked on the specimen before testing, a retest shall be allowed.

Sec. 3.3—Flattening Tests

3.3.1. Nonexpanded electric-welded pipe. For nonexpanded electric-welded pipe (except automatic fusion-welded pipe) produced in single lengths, the crop ends cut from each length shall be flattened between parallel plates, with the weld at the point of maximum bending, until opposite walls of the pipe meet. No opening in the weld shall appear until the distance between the plates is less than two-thirds of the original outside diameter of the pipe, and no cracks or breaks in the metal shall occur elsewhere than in the weld until the distance between the plates is less than one-third of the original outside diameter of the pipe. Also, evidence of laminations or burned metal shall not develop during the entire flattening operation. If any crop end fails to conform to these requirements, additional tests shall be made on specimens cut from the same end of the same length of pipe until the requirements are met, except that the finished pipe shall not be shorter than 80 per cent of its length after the initial cropping. Precautions shall be taken that crop ends can be identified with respect to the length of pipe from which they were cut.

3.3.2. Nonexpanded resistance-welded pipe. For nonexpanded resistance-welded pipe produced in multiple lengths and subsequently cut into single lengths, a flattening test shall be made as described in Sec. 3.3.1 above

on test specimens cut from each end of each coil. If one or both of the specimens fail to conform to the requirements, the manufacturer may elect to make retests on specimens cut from each end of each individual length.

3.3.3. Cold-expanded electric-welded pipe. For cold-expanded electric-welded pipe (except automatic fusion-welded pipe), a flattening test shall be made as described in Sec. 3.3.1 above on a ring 4 in. or wider cut from one end of a length of pipe from each lot of 100 lengths or fewer of each size. If the specimen fails to conform to the requirements, the manufacturer

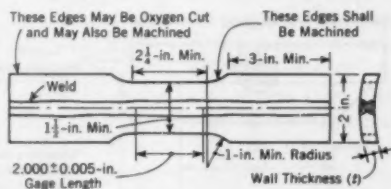


Fig. 1. Tensile-Elongation Test Specimen

The weld reinforcement shall be removed from both faces of the specimen.

may elect to make retests on specimens cut from one end of two additional lengths from the same lot. If both retest specimens conform to the specified requirements, all the lengths in the lot shall be accepted except the length from which the initial specimen was taken. If one or both of the retest specimens fail to conform to the specified requirements, the manufacturer may elect to repeat the test on specimens cut from one end of the individual lengths remaining in the lot.

3.3.4. Butt-welded pipe 2 1/2 in. and larger. For butt-welded pipe 2 1/2 in. and larger, one section of pipe approxi-

mately 6 in. long, cut from a length from each lot of 400 lengths or fewer, of each size, shall be flattened between parallel plates with the weld located

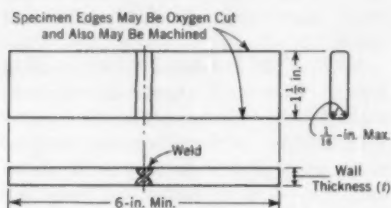


Fig. 2. Guided-Bend Test Specimen

The weld reinforcement shall be removed from both faces of the specimen.

approximately 45 deg from the point of maximum bending. No cracks shall occur in any portion of the pipe, and no opening shall occur in the weld until the distance between the plates is less than 60 per cent of the original outside diameter of the pipe. If the specimen fails to conform to these requirements, the manufacturer may elect to make retests on specimens cut from two additional lengths from the same lot. If all retest specimens conform to the specified requirements, all the lengths in the lot shall be accepted except the length from which the initial specimen was taken. If one or more of the retest specimens fail to

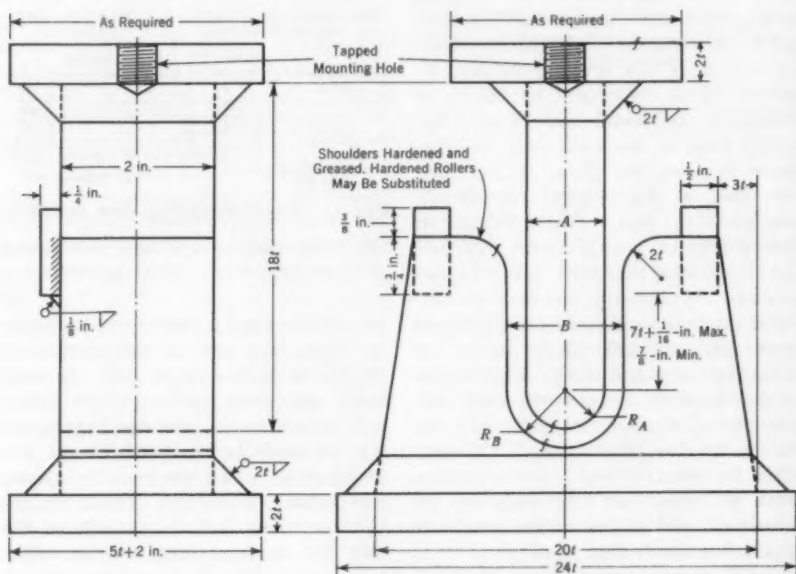


Fig. 3. Jig for Guided-Bend Test

The symbol "f" indicates a light finish cut; t is the tabulated wall thickness of the pipe. For pipe of Grade A or B, the dimensions A and B shall be $4t$ and $6t + \frac{1}{8}$ in., respectively; for pipe of Grade X42, these dimensions shall be $6t$ and $8t + \frac{1}{8}$ in., respectively ($R_A = \frac{1}{2} A$ and $R_B = \frac{1}{2} B$).

conform to the specified requirements, the manufacturer may elect to repeat the test on specimens cut from the individual lengths remaining in the lot.

Sec. 3.4—Bend Tests for Butt-Welded Pipe

For butt-welded pipe 2½ in. and smaller, one full-section specimen of appropriate length, cut from a length of pipe from each lot of 400 lengths or fewer of each size, shall be bent cold through 90 deg around a mandrel having a diameter not greater than twelve times the outside diameter of the pipe being tested, with the weld located approximately 45 deg from the point of contact of the specimen with the mandrel. No cracks shall occur in any portion of the pipe, and no opening shall occur in the weld. If the specimen fails to conform to these requirements, the manufacturer may elect to make retests on specimens cut from two additional lengths from the same lot. If all retest specimens conform to the specified requirements, all the lengths in the lot shall be accepted except the length from which the initial specimen was taken. If one or more of the retest specimens fail to conform to the specified requirements, the manufacturer may elect to repeat the test on specimens cut from the individual lengths remaining in the lot.

Sec. 3.5—Fusion Weld Tests

For fusion-welded pipe, the weld shall be tested by either of the tests described below, at the option of the

manufacturer. The required specimens shall be cut from a length of pipe from each lot of 100 lengths or less of each size. The specimens shall not contain any repair welding made by the manual metallic-arc procedure.

3.5.1. *Tensile-elongation test.* One specimen conforming to Fig. 1 shall be tested in tension, and shall conform to the elongation requirements specified in Table 2.

3.5.2. *Guided-bend test.* One face bend specimen conforming to Fig. 2 shall be bent approximately 180 deg in a jig substantially in accordance with Fig. 3. The specimen shall not fracture completely, and no cracks exceeding ¼ in. in any direction shall be present in the weld metal or between the weld metal and pipe metal. Cracks which originate at the edges of the specimen and which are less than ¼ in. long shall not be cause for rejection.

3.5.3. *Retests.* If the tensile-elongation specimen or the guided-bend specimen fails to conform to the specified requirements, the manufacturer may elect to repeat the test on specimens cut from two additional lengths of pipe from the same lot. If such specimens conform to the specified requirements, all the lengths in the lot shall be accepted, except the length initially selected for the test. If any of the retest specimens fails to pass the specified requirements, the manufacturer may elect to test specimens cut from the individual lengths remaining in the lot. Specimens for retests shall be taken in the same manner as specified in Sec. 3.5.1 and Sec. 3.5.2.

TABLE 5

Dimensions, Weights, and Test Pressures for Mill-Type Steel Water Pipe

Nominal Size in.	Outside Diam. in.	Inside Diam. in.	Weight per Linear Foot* lb	Wall Thickness in.		Butt Welded	Grade A	Grade B
				Decimal	Fraction			
$\frac{1}{8}$	0.405	0.269	0.24†	0.068	$\frac{1}{16}$	700	700	700
$\frac{1}{4}$	0.540	0.364	0.42†	0.088		700	700	700
$\frac{3}{8}$	0.675	0.493	0.57†	0.091	$\frac{3}{32}$	700	700	700
$\frac{1}{2}$	0.840	0.622	0.85†	0.109	$\frac{7}{64}$	700	700	700
$\frac{3}{4}$	1.050	0.824	1.13†	0.113		700	700	700
1	1.315	1.049	1.68†	0.133		700	700	700
$1\frac{1}{4}$	1.660	1.380	2.27†	0.140	$\frac{9}{64}$	800	1,000	1,100
$1\frac{1}{2}$	1.900	1.610	2.72†	0.145		800	1,000	1,100
2	2.375	2.067	3.65†	0.154	$\frac{3}{16}$	800	1,000	1,100
$2\frac{1}{2}$	2.875	2.469	5.79†	0.203	$\frac{13}{64}$	800	1,000	1,100
3	3.500	3.068	7.58†	0.216	$\frac{7}{32}$	800	1,000	1,100
3	3.500	3.250	4.51	0.125	$\frac{1}{2}$		1,300	1,500
		3.188	5.58	0.156	$\frac{5}{32}$		1,600	1,900
		3.124	6.63	0.188	$\frac{3}{16}$		1,900	2,200
		3.068	7.58	0.216	$\frac{7}{32}$		2,200	2,500
		3.000	8.68	0.250	$\frac{1}{2}$		2,500	2,500
		2.938	9.67	0.281	$\frac{9}{32}$		2,500	2,500
$3\frac{1}{2}$	4.000	3.750	5.17	0.125	$\frac{1}{2}$		1,100	1,300
		3.688	6.41	0.156	$\frac{5}{32}$		1,400	1,600
		3.624	7.63	0.188	$\frac{3}{16}$		1,700	2,000
		3.548	9.11†	0.226		1,200	2,000	2,400
		3.500	10.01	0.250	$\frac{1}{2}$		2,200	2,500
		3.438	11.17	0.281	$\frac{9}{32}$		2,500	2,500
4	4.500	4.250	5.84	0.125	$\frac{1}{2}$		1,000	1,200
		4.188	7.25	0.156	$\frac{5}{32}$		1,200	1,500
		4.124	8.64	0.188	$\frac{3}{16}$		1,500	1,800
		4.062	10.00	0.219	$\frac{7}{32}$		1,700	2,000
		4.026	10.79†	0.237	$\frac{13}{64}$		1,900	2,200
		4.000	11.35	0.250	$\frac{1}{2}$		2,000	2,300
		3.938	12.67	0.281	$\frac{9}{32}$		2,200	2,500
		3.876	13.98	0.312	$\frac{5}{16}$		2,500	2,500
5	5.563	5.251	9.02	0.156	$\frac{5}{32}$		1,000	1,200
		5.187	10.76	0.188	$\frac{3}{16}$		1,200	1,400
		5.125	12.49	0.219	$\frac{7}{32}$		1,400	1,700
		5.047	14.62†	0.258			1,700	1,900
		5.001	15.87	0.281	$\frac{9}{32}$		1,800	2,100
		4.939	17.52	0.312	$\frac{5}{16}$		2,000	2,400
		4.875	19.16	0.344	$\frac{11}{32}$		2,200	2,500

* Plain end.

† Standard weight.

TABLE 5—Physical Data for Mill-Type Pipe (contd.)

Nominal Size in.	Outside Diam. in.	Inside Diam. in.	Weight per Linear Foot ^a lb	Wall Thickness in.		Grade A	Grade B
				Decimal	Fractional		
6	6.625	6.249	12.89	0.188	$\frac{3}{16}$	1,000	1,200
		6.187	14.97	0.219	$\frac{7}{32}$	1,200	1,400
		6.125	17.02	0.250	$\frac{1}{4}$	1,400	1,600
		6.065	18.97†	0.280	$\frac{9}{32}$	1,500	1,800
		6.001	21.07	0.312	$\frac{5}{16}$	1,700	2,000
		5.937	23.06	0.344	$\frac{11}{32}$	1,900	2,200
		5.875	25.03	0.375	$\frac{3}{8}$	2,000	2,400
8	8.625	8.249	16.90	0.188	$\frac{3}{16}$	800	900
		8.187	19.64	0.219	$\frac{7}{32}$	900	1,100
		8.125	22.36	0.250	$\frac{1}{4}$	1,000	1,200
		8.071	24.70†	0.277	$\frac{9}{32}$	1,200	1,300
		8.001	27.74	0.312	$\frac{5}{16}$	1,300	1,500
		7.981	28.55†	0.322	$\frac{21}{64}$	1,300	1,600
		7.937	30.40	0.344	$\frac{11}{32}$	1,400	1,700
		7.875	33.04	0.375	$\frac{3}{8}$	1,600	1,800
		7.749	38.26	0.438	$\frac{7}{16}$	1,800	2,100
10	10.750	10.374	21.15	0.188	$\frac{3}{16}$	650	750
		10.312	24.60	0.219	$\frac{7}{32}$	750	850
		10.250	28.04	0.250	$\frac{1}{4}$	850	1,000
		10.192	31.20†	0.279	$\frac{9}{32}$	1,000	1,200
		10.136	34.24†	0.307	$\frac{5}{16}$	1,000	1,200
		10.062	38.20	0.344	$\frac{11}{32}$	1,100	1,300
		10.020	40.48†	0.365	$\frac{7}{16}$	1,200	1,400
		9.874	48.19	0.438	$\frac{7}{16}$	1,500	1,700
12	12.750	12.312	29.28	0.219	$\frac{7}{32}$	600	700
		12.250	33.38	0.250	$\frac{1}{4}$	700	800
		12.188	37.45	0.281	$\frac{9}{32}$	800	950
		12.126	41.51	0.312	$\frac{5}{16}$	900	1,000
		12.090	43.77†	0.330	$\frac{11}{32}$	1,000	1,200
		12.062	45.55	0.344	$\frac{3}{8}$	1,000	1,200
		12.000	49.56†	0.375	$\frac{3}{8}$	1,100	1,200
		11.938	53.56	0.406	$\frac{13}{32}$	1,100	1,300
		11.874	57.53	0.438	$\frac{7}{16}$	1,200	1,400
14	14.000	13.500	36.71	0.250	$\frac{1}{4}$	650	750
		13.438	41.21	0.281	$\frac{9}{32}$	700	850
		13.376	45.68	0.312	$\frac{5}{16}$	800	950
		13.312	50.14	0.344	$\frac{11}{32}$	900	1,000
		13.250	54.57†	0.375	$\frac{3}{8}$	950	1,100
		13.124	63.37	0.438	$\frac{7}{16}$	1,100	1,300
		13.000	72.09	0.500	$\frac{1}{2}$	1,300	1,500
16	16.000	15.500	42.05	0.250	$\frac{1}{4}$	550	650
		15.438	47.22	0.281	$\frac{9}{32}$	650	750
		15.376	52.36	0.312	$\frac{5}{16}$	700	800
		15.312	57.48	0.344	$\frac{11}{32}$	750	900
		15.250	62.58†	0.375	$\frac{3}{8}$	850	1,000
		15.124	72.72	0.438	$\frac{7}{16}$	1,000	1,100
		15.000	82.77	0.500	$\frac{1}{2}$	1,100	1,300

^a Plain end.

† Standard weight.

TABLE 5—Physical Data for Mill-Type Pipe (contd.)

Nominal Size in.	Outside Diam. in.	Inside Diam. in.	Weight per Linear Foot* lb	Wall Thickness in.		Grade A	Grade B	Grade X42
				Decimal	Fractional			
18	18.000	17.500	47.39	0.250	$\frac{1}{4}$	500	600	
		17.438	53.22	0.281	$\frac{3}{32}$	550	650	
		17.376	59.03	0.312	$\frac{1}{8}$	600	750	
		17.312	64.82	0.344	$\frac{3}{16}$	700	800	
		17.250	70.59†	0.375	$\frac{1}{2}$	750	900	
		17.124	82.06	0.438	$\frac{3}{8}$	900	1,000	
		17.000	93.45	0.500	$\frac{1}{2}$	1,000	1,200	
20	20.000	19.500	52.73	0.250	$\frac{1}{4}$	450	500	
		19.438	59.23	0.281	$\frac{3}{32}$	500	600	
		19.376	65.71	0.312	$\frac{1}{8}$	550	650	
		19.312	72.16	0.344	$\frac{3}{16}$	600	700	
		19.250	78.60†	0.375	$\frac{1}{2}$	700	800	
		19.124	91.41	0.438	$\frac{3}{8}$	800	900	
		19.000	104.13	0.500	$\frac{1}{2}$	900	1,000	
22	22.000	21.376	72.38	0.312	$\frac{1}{8}$	500	600	
		21.312	79.51	0.344	$\frac{3}{16}$	550	650	
		21.250	86.61	0.375	$\frac{1}{2}$	600	700	
		21.124	100.75	0.438	$\frac{3}{8}$	700	850	
		21.000	114.81	0.500	$\frac{1}{2}$	800	950	
24	24.000	23.500	63.41	0.250	$\frac{1}{4}$	400	450	790
		23.438	71.25	0.281	$\frac{3}{32}$	400	500	890
		23.376	79.06	0.312	$\frac{1}{8}$	450	550	990
		23.312	86.85	0.344	$\frac{3}{16}$	500	600	1,090
		23.250	94.62	0.375	$\frac{1}{2}$	550	650	1,190
		23.188	102.37	0.406	$\frac{5}{16}$	600	700	1,280
		23.124	110.10	0.438	$\frac{3}{8}$	650	750	1,380
		23.000	125.49	0.500	$\frac{1}{2}$	750	850	1,580
26	26.000	25.500	68.75	0.250	$\frac{1}{4}$			730
		25.438	77.25	0.281	$\frac{3}{32}$			820
		25.376	85.73	0.312	$\frac{1}{8}$			910
		25.312	94.19	0.344	$\frac{3}{16}$			1,010
		25.250	102.63	0.375	$\frac{1}{2}$			1,100
		25.188	111.05	0.406	$\frac{5}{16}$			1,190
		25.124	119.44	0.438	$\frac{3}{8}$			1,280
		25.062	127.82	0.469	$\frac{11}{16}$			1,370
		25.000	136.17	0.500	$\frac{1}{2}$			1,460
28	28.000	27.500	74.09	0.250	$\frac{1}{4}$			680
		27.438	83.26	0.281	$\frac{3}{32}$			760
		27.376	92.41	0.312	$\frac{1}{8}$			850
		27.312	101.53	0.344	$\frac{3}{16}$			930
		27.250	110.64	0.375	$\frac{1}{2}$			1,020
		27.188	119.72	0.406	$\frac{5}{16}$			1,100
		27.124	128.79	0.438	$\frac{3}{8}$			1,190
		27.062	137.83	0.469	$\frac{11}{16}$			1,270
		27.000	146.85	0.500	$\frac{1}{2}$			1,360

* Plain end.

† Standard weight.

TABLE 5—Physical Data for Mill-Type Pipe (contd.)

Nominal Size in.	Outside Diam. in.	Inside Diam. in.	Weight per Linear Foot* lb	Wall Thickness in.		Grade X42
				Decimal	Fractional	Test Pressure psi
30	30.000	29.500	79.43	0.250	$\frac{1}{4}$	630
		29.438	89.27	0.281	$\frac{5}{16}$	710
		29.376	99.08	0.312	$\frac{3}{8}$	790
		29.312	108.88	0.344	$\frac{7}{16}$	870
		29.250	118.65	0.375	$\frac{1}{2}$	950
		29.188	128.40	0.406	$\frac{5}{8}$	1,030
		29.124	138.13	0.438	$\frac{3}{4}$	1,110
		29.062	147.84	0.469	$\frac{7}{8}$	1,190
		29.000	157.53	0.500	$\frac{1}{2}$	1,270
32	32.000	31.500	84.77	0.250	$\frac{1}{4}$	600
		31.438	95.28	0.281	$\frac{5}{16}$	670
		31.376	105.76	0.312	$\frac{3}{8}$	740
		31.312	116.22	0.344	$\frac{7}{16}$	820
		31.250	126.66	0.375	$\frac{1}{2}$	890
		31.188	137.08	0.406	$\frac{5}{8}$	960
		31.124	147.48	0.438	$\frac{3}{4}$	1,040
		31.062	157.86	0.469	$\frac{7}{8}$	1,110
		31.000	168.21	0.500	$\frac{1}{2}$	1,190
34	34.000	33.500	90.11	0.250	$\frac{1}{4}$	560
		33.438	101.28	0.281	$\frac{5}{16}$	630
		33.376	112.43	0.312	$\frac{3}{8}$	700
		33.312	123.56	0.344	$\frac{7}{16}$	770
		33.250	134.67	0.375	$\frac{1}{2}$	840
		33.188	145.76	0.406	$\frac{5}{8}$	910
		33.124	156.82	0.438	$\frac{3}{4}$	980
		33.062	167.87	0.469	$\frac{7}{8}$	1,050
		33.000	178.89	0.500	$\frac{1}{2}$	1,120
36	36.000	35.500	95.45	0.250	$\frac{1}{4}$	530
		35.438	107.29	0.281	$\frac{5}{16}$	600
		35.376	119.11	0.312	$\frac{3}{8}$	660
		35.312	130.90	0.344	$\frac{7}{16}$	730
		35.250	142.68	0.375	$\frac{1}{2}$	790
		35.188	154.43	0.406	$\frac{5}{8}$	860
		35.124	166.17	0.438	$\frac{3}{4}$	930
		35.062	177.88	0.469	$\frac{7}{8}$	990
		35.000	189.57	0.500	$\frac{1}{2}$	1,060

* Plain end.

Section 4—Hydrostatic Tests

Sec. 4.1—Mill Inspection Hydrostatic Test

Each length of pipe shall withstand, without leakage, a mill inspection hydrostatic test to at least the pressure shown in Table 5. Sizes larger than shown in Table 5 shall be tested to at least the pressure specified by the purchaser. Test pressures for all sizes of seamless pipe and for welded pipe in sizes 18 in. and smaller shall be held for not less than 5 sec. Test pressures for welded pipe in sizes 20 in. and larger shall be held for not less than 10 sec. For threaded-and-coupled pipe, the test shall be applied with the

couplings made handling-tight, or made up power-tight if power-tight makeup is specified on the purchase order, except that 20-in. pipe and larger may be tested in the plain-end condition.

Sec. 4.2—Special Sections

4.2.1. Special sections and fittings made from pipe manufactured to this standard need not be hydrostatically tested.

4.2.2. All special sections and fittings not made from pipe manufactured to this standard shall be subjected to a hydrostatic test at a pressure* specified by the purchaser.

Section 5—Circumferential Welding of Straight Pipe Lengths

Sec. 5.1—Method

Welding of any type generally recognized as sound commercial practice shall be permitted.

Sec. 5.2—Workmanship

5.2.1. The ends of the pipe to be welded together shall be prepared in accordance with the requirements of the process used. The completed length shall be straight, within limits specified in this standard.

5.2.2. The weld shall have a substantially uniform cross section throughout the circumference of the pipe. The crowned surface shall at no point be below the outside surface of the parent metal, nor shall the crown at any point rise above the outside surface of the pipe more than $\frac{1}{8}$ in. Finished lengths of mill pipe, when so stipulated, shall permit the passage of a drift $\frac{1}{8}$ in. less than the nominal inside diameter of the pipe for sizes up to but not including $8\frac{1}{2}$ in.; $3\frac{5}{8}$ in. on sizes $8\frac{1}{2}$ –14 in.; and $\frac{3}{4}$ in. on sizes 16–20 in.

Section 6—Manual Metallic-Arc Repair Welding

Sec. 6.1—Procedure

All manual metallic-arc repair welds shall be made according to a tested procedure and by a repair welder qualified in accordance with Sec. 6.2 and Sec. 6.3, except that, at the option of the manufacturer, the test specified in the latest edition of the following publications may be substituted: API

1104 (Standard for Field Welding of Pipelines), ASA B31.1 (American Standard Code for Pressure Piping, Sec. 6), and ASME Boiler and Pres-

*The hydrostatic test pressures are mill inspection test pressures. They are not intended as a basis for design, and do not necessarily have any direct relationship to working pressures.

sure Vessel Code, Sec. IX. Test welds may be made either on plate stock or on pipe stock at the option of the manufacturer. The manufacturer shall maintain a record of the procedure and performance test results.

Sec. 6.2—Procedure Tests

Manual metallic-arc repair welding procedure tests shall be required on two specimens of every grade and on material at least as thick as the pipe on which repair welds are to be made. The transverse butt weld tensile test and either the longitudinal weld tensile-elongation test or the transverse guided weld bend test shall be made, except that the manufacturer

conform to Fig. 4. The weld shall be made in a groove as shown. The elongation after complete rupture of the test specimen in tension shall be at least equal to the minimum elongation specified for the grade.

6.2.3. *Transverse guided weld bend test.* The transverse guided weld bend test specimen shall conform to Fig. 5. The weld shall be made in a groove as shown. Specimens shall be bent approximately 180 deg in a jig substantially in accordance with Fig. 3 with the exposed surface of the weld in tension. The bend test shall be acceptable if no crack or other defect exceeding $\frac{1}{8}$ in. in any direction is present in the weld metal or between

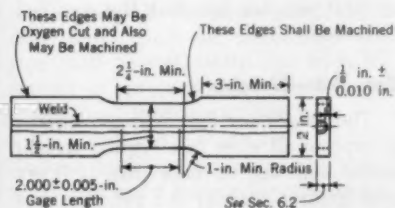


Fig. 4. Tensile-Elongation Test Specimen

The weld reinforcement shall be removed.

shall have the option set forth in Sec. 6.1 above.

6.2.1. *Transverse butt-weld tensile test.* The transverse butt-weld tensile test specimen shall be approximately $1\frac{1}{2}$ in. wide and shall have the transverse manual metallic-arc butt weld perpendicular to the longitudinal axis at the center of the test specimen. The weld reinforcement shall be removed from both faces. The ultimate tensile strength shall be at least equal to the minimum specified for the grade.

6.2.2. *Longitudinal-weld tensile-elongation test.* The longitudinal-weld tensile-elongation test specimen shall

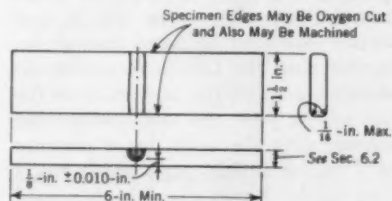


Fig. 5. Guided-Bend Test Specimen

The weld reinforcement shall be removed.

the weld and the pipe metal after bending. Cracks which originate along the edges of the specimen during testing and which are less than $\frac{1}{8}$ in. measured in any direction shall not be considered detrimental.

Sec. 6.3—Repair Welder's Performance Tests

Manual metallic-arc repair welder's performance tests shall be required on two specimens of every grade. If either of the two specimens fails to conform to the requirements specified, four retests shall be required immediately or two retests shall be required if the welder takes further instruction

in the practice before making a retest. All retests shall conform to the requirements specified. Further performance tests are not required unless the repair welder has not engaged in the tested repair welding procedure for a period of 3 months or more,

or unless there is some specific reason to question his ability. Either of the following tests shall be made: the longitudinal weld tensile-elongation test (described in Sec. 6.2.2, above) or the transverse guided-weld bend test (described in Sec. 6.2.3, above).

Section 7—Tolerances

Sec. 7.1—Diameter

The outside diameter of pipe 1.900 in. and smaller shall not be more than 0.016 in. larger or 0.031 in. smaller than the specified outside diameter. The outside diameter of pipe larger than 1.900 in. shall not vary from the specified outside diameter by more than 1.0 per cent. Pipe 10 $\frac{3}{4}$ in. and smaller shall not be more than $\frac{1}{8}$ in. smaller than the tabulated outside diameter for a distance of 4 in. from the end of the pipe and shall permit the passage over the ends, for a distance of 4 in., of a ring gage which has a bore $\frac{1}{8}$ in. larger than the tabulated outside diameter of the pipe. Pipe 12 $\frac{3}{4}$ –20 in. shall not be more than $\frac{1}{8}$ in. smaller than the tabulated outside diameter for a distance of 4 in. from the end of the pipe and shall permit the passage over the ends, for a distance of 4 in., of a ring gage which has a bore $\frac{3}{8}$ in. larger than the tabulated outside diameter of the pipe. At the option of the manufacturer, the minimum outside diameter of pipe in sizes 20 in. and smaller may be based on circumferential measurements. Pipe 22 in. and larger shall not be more than $\frac{1}{8}$ in. smaller nor more than $\frac{3}{8}$ in. larger than the tabulated outside diameter for a distance of 4 in. from the end of the pipe, based on circumferential measurements.

Sec. 7.2—Wall Thickness

For pipe in sizes 18 in. and smaller, the wall thickness shall not be more than 15.0 per cent greater than or 12.5 per cent less than the specified thickness. For pipe in sizes 20 in. and larger the wall thickness shall not be more than 15.0 per cent greater than or 10.0 per cent less than the specified thickness.

Sec. 7.3—Weight

The weight per foot of single lengths of pipe of all sizes in a regular-weight series shall not be more than 10.0 per cent greater than or 3.5 per cent less than the specified weight. The weight of carload lots of pipe in all sizes shall not be more than 1.75 per cent less than the specified weight.

Sec. 7.4—Length

7.4.1. *Straight pipe sections.* For all classes of pipe, when ordered to a specified length, the actual length shall not vary from the specified length by more than $\frac{1}{8}$ in. Unless otherwise agreed upon between the purchaser and the manufacturer, pipe shall be furnished in single random lengths or double random lengths as specified on the purchase order. If single random lengths are specified, the average length shall not be less than 17 $\frac{1}{2}$ ft, and no

piece shall be shorter than 9 ft. If double random lengths are specified, the average length shall not be less than 35 ft, not more than 10 per cent of the lengths shall be shorter than $26\frac{1}{4}$ ft, and no piece shall be shorter than 14 ft. If the average length agreed upon is in excess of 20 ft, not more than 10 per cent of the lengths shall be shorter than 75 per cent of the average length agreed upon, and no length shall

be shorter than 40 per cent of the average length agreed upon.

7.4.2. Special sections. For special sections, such as elbows, wyes, tees, reducers, and closure pieces, the length tolerance shall be that shown in the drawings.

Sec. 7.5—Straightness

Finished pipe shall be reasonably straight for the application of interior lining.

Section 8—Pipe Ends

Sec. 8.1—Ends for Mechanical Couplings

8.1.1. Plain ends for use with mechanical couplings shall be square cut or beveled and the inside and outside edge burrs shall be removed. The pipe shall be sufficiently free from indentations, projections, or roll marks at each end to make a tight joint.

8.1.2. Ends for other types of mechanical couplings shall have tolerances within the limits required by the manufacturer of the coupling to be used.

Sec. 8.2—Ends for Field Butt Welding

8.2.1. For pipe of wall thickness $\frac{1}{4}$ in. or greater for field butt welding, the ends shall be beveled on the outside, on the inside, or from both sides, as specified, to an angle of 30 deg, with a maximum plus tolerance of 5 deg and no minus tolerance. It shall be understood that the angle of the bevel is measured from a line drawn perpendicular to the axis of the pipe. The width of the root face at the end of the pipe shall be $\frac{1}{16}$ in. with a tolerance of $\frac{1}{32}$ in.

8.2.2. For pipe of wall thickness less than $\frac{1}{4}$ in. for field butt welding, the ends need not be beveled.

Sec. 8.3—Bell-and-Spigot Ends With Rubber Gasket

For bell-and-spigot ends for use with a rubber gasket, the difference in measurement between the outside circumference of the spigot and the inside circumference of the bell shall be no less than 0.060 in. and no greater than 0.250 in.

Sec. 8.4—Threaded Ends

Threaded ends shall conform to the threading requirements specified in Sec. IX of API 5L. One end of each length of threaded pipe shall be provided with a coupling, and the other end with thread protection, both conforming to the requirements of Sec. VIII of API 5L. Couplings shall be screwed to the pipe handling-tight except that they shall be applied power-tight if so specified on the purchase order. A high-grade thread compound shall be applied to cover the full surface of either the coupling or pipe-engaged thread before making up the joint. All exposed threads shall be coated with a high-grade thread compound unless otherwise specified on the purchase order.

Sec. 8.5—Ends for Other Types of Joints

Ends for types of joints not covered above shall be made in accordance with the approved drawings. The other types of ends that may be used are:

ends with slip joints (lap joints) for field welding, ends fitted with butt straps for field welding, ends for riveted field joints, bell-and-spigot ends for calked joints, plain ends fitted with flanges, and tapered ends for driven field joints.

Section 9—Marking and Delivery**Sec. 9.1—Marking**

Each length of pipe shall be legibly marked by paint stenciling, die stamping, or hot-roll marking to show the manufacturer's name or mark, size of pipe, weight per foot, and grade (whether A, B, or X42).

Sec. 9.2—Delivery

Subject to agreement between the purchaser and the manufacturer, pipe sections shall be delivered in the order quired and shall be loaded in accordance with specifications set forth by the purchaser.

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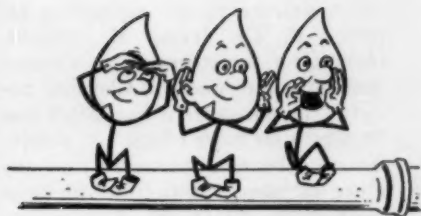
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Percolation and Runoff

A&P water—Atlantic and Pacific water (with the salt removed), that is—may not be available at super-market prices yet, but it is being super-marketed these days by the Office of Saline Water and the industries co-operating with it in its effort to find ways to desalt sea water at a cost approaching that of developing present fresh-water supplies.

Helping to expose the public to the thought that the sea soon not only can, but *must* supply a considerable portion of our fresh-water requirements have been all manner of media. In recent months, the *Wall Street Journal*, the *New York Herald-Tribune*, the *Baltimore Sun*, the *Providence Journal*, the *League of Women Voters' National Voter*, *American Legion* and *Look* magazines, and other publications too numerous to mention have surveyed the subject. And in almost any edition of almost any newspaper these days there is some sort of reference to "fresh water from the sea."

In all this wealth of water words there is not much that would be new to the JOURNAL reader. Announcement of a "revolutionary" machine developed by Chance Vought Aircraft Inc. to "utilize radioisotope energy" in

a distillation process, reports of a new desalting filter developed by two University of California professors using membranes produced by mixing cellulose acetate with a water solution of magnesium perchlorate and acetone, and a description of the groundbreaking, at Freeport, Tex., for the first of the Office of Saline Water's five 1-mgd experimental facilities were the only real "news" items we could cull from a mountain of clippings and communications. But new or not to us, these words are serving to educate the public not only in the importance of water supply, but in the fact that water supply does cost money. Thus, though we're not about to buy the A&P product, we certainly are happy to have it in such prominent prospect.

Good Gulf water, meanwhile, is already being marketed on a commercial basis not only not desalted, but condensed. Rathbun Co., industrial chemicals firm of Houston, Tex., packages Gulf water, containing all 44 of its chemicals, in polyethylene containers and sells it at \$4.50 per gallon in Houston or \$5.50 per gallon postpaid. Customers are sufferers of arthritis and other rheumatic conditions, to

(Continued on page 40 P&R)

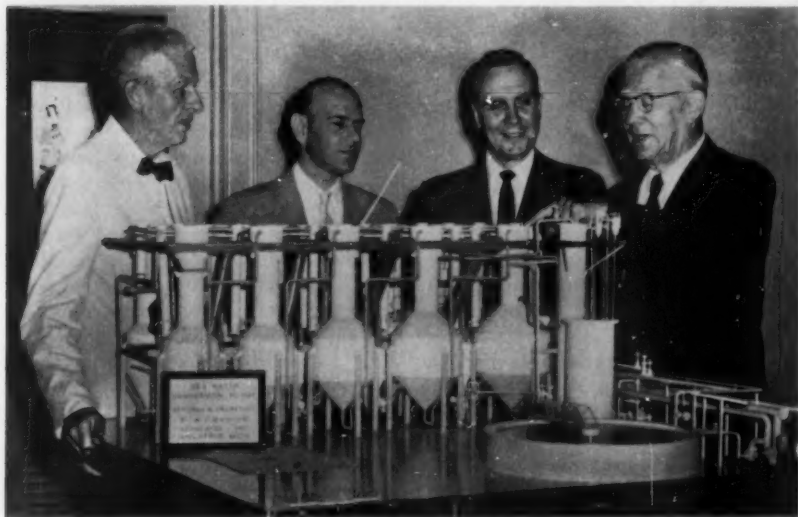
(Continued from page 39 P&R)

whom, medical researchers have recently discovered, consumption of sea water can be beneficial.

Even without a report on the market potential or on the cost of polyethylene bags, we have an idea that a \$4.50 per gallon byproduct might be welcome to the producers of a product that costs \$1 per 1,000 gal.

Debubblification of water supplies may at last be possible if a new ion exchanger called "Duolite" (A-102D), manufactured by the Chemical Process Co. of Redwood City, Calif., measures

up in practice to the capabilities reported by Dr. Irving M. Abrams, chemist for the company, at a recent meeting of the American Chemical Society. In the paper it was noted that Duolite is far more effective in removing alkyl benzene sulfonates than any other exchanger, 1 cu ft of the resin cleaning up 750,000 gal of water containing 2 ppm ABS. Yet to be worked out, according to the report, are the economics of recharging the resin, which Dr. Abrams noted would probably not be accomplished "until removal of the detergent contamination



Object of attention is a scale model of the 1-mgd long-tube vertical multiple-effect distillation plant for conversion of sea water that will be built at Freeport, Tex., as the first of five experimental plants authorized by the Department of Interior's Office of Saline Water. Designed and engineered by W. L. Badger Assoc. of Ann Arbor, Mich., the plant will be constructed by Chicago Bridge & Iron Co. Shown examining the model are Joseph L. Miller, CB&I, Allan Cywyn, OSW, S. L. Luce, CB&I, and A. L. Miller, OSW director. Ground was broken for the plant on Aug. 30 by Secretary of the Interior Fred A. Seaton.

(Continued on page 42 P&R)



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APPROVED FOR DRINKING WATER BY NATIONAL SANITATION FOUNDATION

Orangeburg Manufacturing Co., Orangeburg, New York, Division of The Flintkote Company, Manufacturer of America's Broadest Line of Building Products.



(Continued from page 40 P&R)

becomes more an urgent need than it is today." He's thinking about health needs, of course, but he should realize that live customers complain even more than dead ones.

Among water's powers, so often and extensively extolled by us, we may very well have forgotten "happiness," which, though difficult to meter, is certainly at every tap. We may have mentioned the relaxation and sense of well-being that come with a hot tub or a brisk shower, but we rather doubt that we have called attention to the happiness of "the man who holds the hose." That was the subject of an editorial in the Aug. 12 edition of the Charlotte, N.C., *News*, which we reprint here without permission, but with great appreciation:

The man who pilots an airplane or drives a bulldozer or wields a sledge hammer may experience an intoxicating sense of power, but on a hot summer afternoon in the garden, the man who holds a hose is the happiest man we know—the man we'd like most to be.

You have all the satisfaction of feeling useful without any of the discomforts of dirty grubbing with hoe or mattock. Like a king, you are monarch of all you survey—the grass, the shrubs, the zinnias, the marigolds, and the phlox. Whatever you decide to water will gratefully receive your attention and respond by living, growing, blooming.

It's the easiest job in the garden—no awkward squatting, no painful stooping to cause tired muscles or aching back. You just hold the hose and let it squirt, with an occasional flick of the wrist to change its direction.

You are free to let your eyes wander around the garden in admiration of your own handiwork—or you can let your mind wander off into reminiscences of

childhood or off to imagine vacations at the beach or the mountains. If the weather is really hot, you can arrange your spraying to cool off your own head between flower beds.

Best of all, while you hold the hose, you give the impression of really working. If your wife wants the floors waxed or the garbage emptied, she has to wait until you are through watering the garden. If your daughter wants to be taken somewhere in the family car, she, too, can just wait until you finish that watering project.

Some people think that riding horseback is the ultimate in sitting in the seat of power. Others might vote for the top of a ferris wheel as it revolves and lifts you toward the heavens, or a pair of water skis as you glide magnificently through the air.

But the easiest and laziest way to get that feeling of owning the world is to stand in your own backyard on a summer afternoon with a garden hose in your hand.

The National Institutes of Health of USPHS have announced a grant award of \$56,571 to Paul E. Morgan, associate professor of civil engineering, and E. Robert Baumann, professor of civil engineering of Iowa State University. The grant will finance a 3-year study of design requirements for municipal diatomite filters. Baumann has returned to the university after completing a year of postdoctoral study under a National Science Foundation fellowship at the University of Durham, Newcastle on Tyne, England.

ASTM has selected Thomas A. Marshall Jr. as its new executive secretary. Formerly senior assistant secretary of ASME, he replaces Robert J. Painter, who will continue to serve the society as consultant.

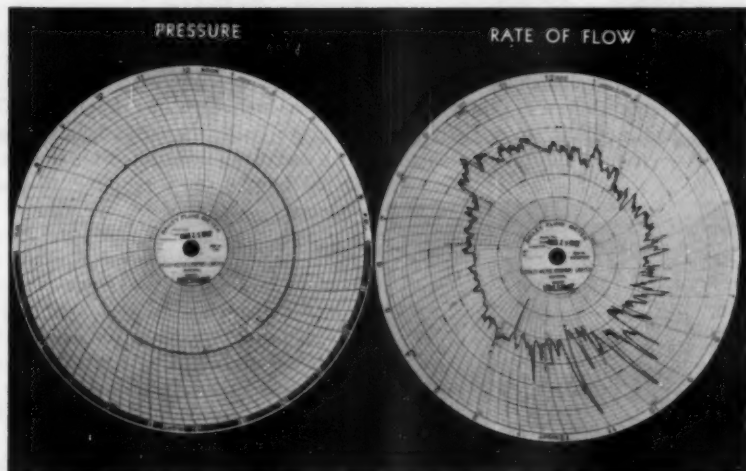
(Continued on page 44 P&R)

Edmonton depends on

Bailey!

Every hour of every day Edmonton, Alberta's waterworks maintains a constant water pressure... and BAILEY does it!

The Bailey Air-Operated Control System automatically regulates two 15,000,000 Imperial Gallon capacity, variable speed, high lift pumps each fitted with a hydraulic coupling...



Here are the Pressure and Rate of Flow records from one of these high lift pumps. Note what happened at 10:55 p.m. on August 25th... the flow increased 10.2 M.I.G.P.D. (67% of pump capacity) almost instantaneously, and there was a momentary drop in pressure of only 5 psi!

During the 24 hours when these charts were being recorded only one variable speed pump was in operation... and its output represented approximately 20% of the total plant output.

Every minute of every hour, Edmonton's water supply depends on BAILEY!

W-7.5

WATER & WASTE TREATMENT DIVISION
BAILEY METER COMPANY

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In Canada — Bailey Meter Company Limited, Montreal



(Continued from page 42 P&R)

'Prewar' is the way Illinois Section Secretary Dewey Johnson is describing the exhumed water pipe in the picture below to Antoinette Woodzick, identified as a "side-walk superintendent." What Dewey hasn't admitted, of course, is that he means pre-The War Between the States, the 4-in. main having been laid in Randolph St., Chicago, in 1852, after the state legislature authorized the city to form the Chicago City Hydraulic Co. to improve the city's supply and distribution system. Unfortunately, Dewey's posing with the girl had nothing to do with his AWWA activity; he is incidentally



also director of research for the Cast Iron Pipe Research Assn. The pipe? Why it's . . . still in good shape says Dewey.

Ice water stories, no doubt calculated to prepare us for an early winter, have been coming from all directions of late:

From the village of Bunji in north-east Pakistan, we have news of the successful planting of a glacier that will provide a regular water supply to irrigate the arid lands of the region. The seed used was a 1,500-lb chunk

of an existing glacier. Chipped off with ice axes, this chunk was carried by relays of ten-man teams across 10 miles of rough country to the selected site, at an altitude of 16,000 ft. Planting involved digging a pit into which saltpeter, hay, and local herbs were first strewn to reduce soil temperature. The ice was then set in and, during the winter, became deeply rooted. Now it will grow each year, insuring not only a regular, but growing supply of fresh water for the growing seasons. Cost of the job was \$300 and two bulls, the latter being offered as a sacrifice (substituted for the traditional human one) required to guarantee the success of the operation.

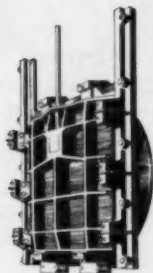
From Lambarene, French Equatorial Africa, we have news of the arrival of an ice wagon from Norway which delivered a 5,400-lb chunk after a 7,500-mile trip that involved 4,300 miles of driving in Africa, including a crossing of the Sahara. Purpose of this trek, which produced only 336 liters of water despite temperatures up to 122° F, was not to provide the means of cooling Dr. Schweitzer's martinis, but to prove the effectiveness of an unidentified insulating material. Having started at Mo-I-Rana, in northern Norway, on Feb. 22, the delivery required just a little more than a month ending in Lambarene on Mar. 24 with all but 10 per cent of the 3 tons of polar ice loaded near the Arctic Circle. Either because no one had then heard of the Bunji business (above) or because the natives owned no skis, no effort at planting was made.

From Juneau, Alaska, in mid-August came word that the Nenana Ice Pool would be preserved, the state's new law on gambling providing for its legalization. As the pool is fed

(Continued on page 46 P&R)

Armco's Complete Line of Gates...

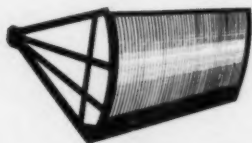
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Flap



Roller



Radial



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water control need

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The four basic types of Armco Gates, as shown above, are available for a variety of waterway openings and water heads. You have a wide choice to meet your specific requirements. Armco Slide Gates are designed for maximum face heads up to 100 feet; Flap Gates permit free outflow while restricting backflow; Radial and Roller Gates are for wide waterway openings near the surface. Armco Gates will do the job you want. Write us for complete information: Armco Drainage & Metal Products, Inc., 5840 Curtis Street, Middletown, Ohio.

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(Continued from page 44 P&R)

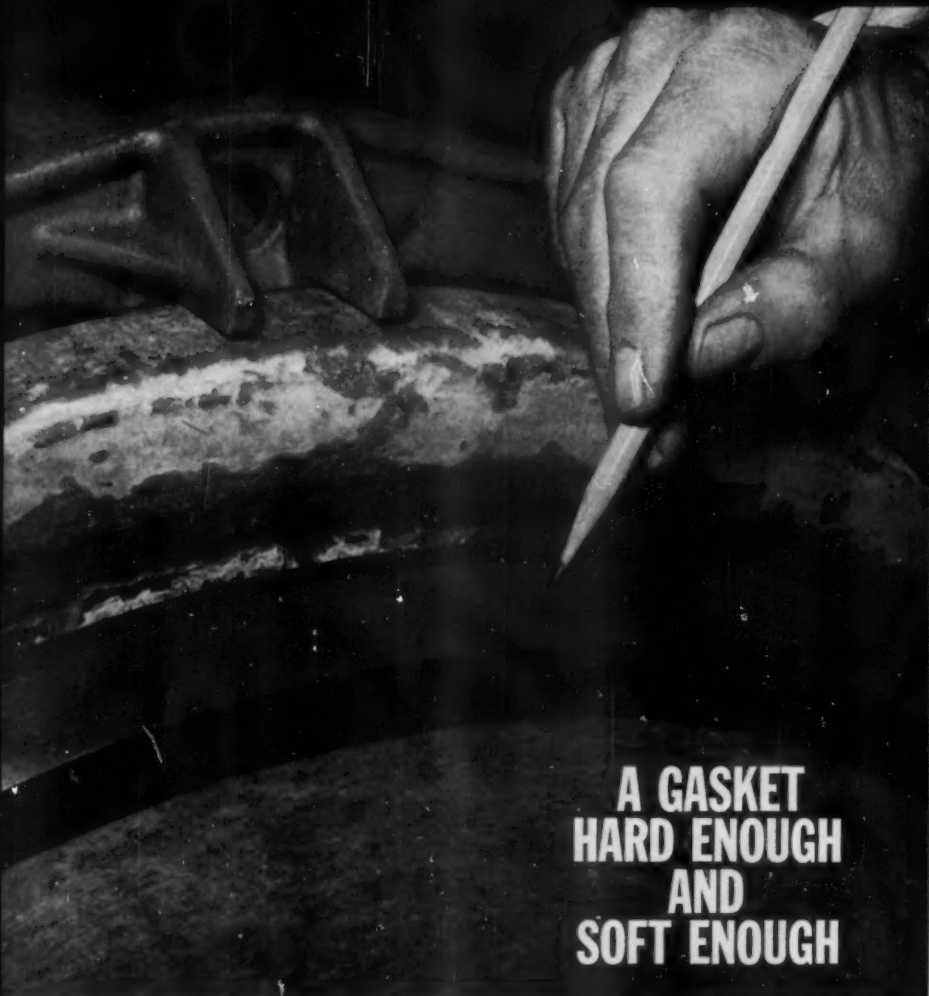
by \$1-per-ticket guesses on the minute that winter ice will break up at Nenana on the Tanana River below Fairbanks and as first prize is often in the neighborhood of \$100,000, at Nenana as at Lambarene, the water in the ice is strictly incidental.

From Fort Belvoir, Va., however, has come a real icewater word in the news that the soldiers and scientists who will spend next winter on Greenland's ice cap will drink water from wells drilled through the snow and ice with steam. The bits employed in the drilling job use steam at 377°F, one drilling vertically to a depth of about 150 ft, or until homogeneous ice structure is reached, the second shooting jets of steam horizontally to melt a bell-shaped reservoir up to 50 ft in diameter, from which water is pumped. Incidentally, as the Army Engineer Research and Development staff, which developed the technique, has no imagination, the steam for the work is not being trucked to Greenland from Old Faithful, but is being supplied by a portable generator made by Vapor Heating Corp., Chicago, for steam cleaning heavy railroad equipment.

From New York City, it was the other end of the water business that introduced ice in August to save the city \$1,000,000 in the construction of a sewage main access shaft by freezing the earth around the drilling area. Here the ice was home grown, not from seed, but from brine poured into pipe driven 300 ft into the earth around the shaft location.

From an Army base in Germany came word of an order issued during a local typhoid epidemic, directing that "all ice cubes will be boiled before using."

(Continued on page 48 P&R)

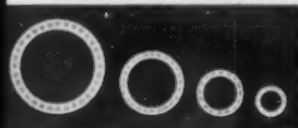


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(Continued from page 46 P&R)

And from London, England, having nothing at all to do with water, United Press International reported that a frozen-food distributing company was advertising its product as "the finest food you ever thaw."

The Maxim Division has been purchased from Emhart Manufacturing Co. by American Machine & Foundry Co. Maxim products, including its desalinization equipment, will continue to be manufactured, by J. B. Beaird Co., Inc., Shreveport, La., a subsidiary of AMF.

'Philadelphia water,' a term once polluted with the unhappiest of connotations, has in the last few years com-

pletely lost its sting. In fact, under the ministrations of Commissioner Sam Baxter, General Superintendent Gerry Arnold, and Chief of Water Operations Vic Appleyard, "Philadelphia water" has grown into an antonym of itself. Thus, when Gore Vidal in his new Broadway play, *The Best Man*, raked up the past by having one of his Presidential aspirants advise his wife not to drink "Philadelphia water," he aroused what the New York *Herald-Tribune* termed "the city solons," who immediately protested, pointing out that Philadelphia had the best filtration plant east of the Mississippi and was universally praised by the health authorities for the quality of its water. The protest,

(Continued on page 50 P&R)

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(Continued from page 48 P&R)

of course, gained more publicity than the gag, and as a result not only the present high quality, but the present local pride in that quality, received a wide airing. Gore Vidal may not change his lines, but we are happy to confirm that Philadelphians and their guests have most assuredly changed their tune—"Philadelphia water" these days is hard to beat!

Glenn W. Holcomb, head of the department of civil engineering, Oregon State College, Corvallis, Ore., has been elected president of ASCE. He replaces Frank A. Marston, consulting engineer of Boston.

The finer things, we have long maintained, are not appreciated by our communist contraparts. From the recent histrionics at the UN, however, we cannot overlook the fact that the *finest thing* apparently is. At any rate, the report of the United Features Syndicate would certainly seem to indicate that in noting:

After 20 minutes of invective against the "piracy" practiced by the United States he was thirsty and sipped appreciatively from a glass of what he told his viewers was "excellent Soviet water," which he recommended to all.

From the way most of our experiences turn out, though, we should probably assume it was really vodka.

The National Conference on Water Pollution will be sponsored by USPHS on Dec. 12-14, at the Sheraton-Park Hotel, Washington, D.C. This will be the first meeting of this nature to be sponsored by the federal government. (Details of the program appear in this month's *Willing Water*.)

(Continued on page 52 P&R)

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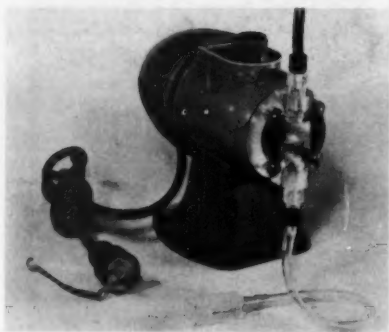
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(Continued from page 50 P&R)



A solution feeder for use on small water systems has been introduced by Calgon Co., division of Hagan Chemicals & Controls, Inc. The new feeder (see cut) is designed for use in home and commercial well systems, cooling towers, and swimming pools. It feeds hypochlorite to well systems and feeds phosphate to correct difficulties with iron, control corrosion, or prevent scale formation.

The World Health Organization will provide in 1961 a limited number of short-term fellowships for foreign study.

Applicants must be US citizens, engaged in full-time public health or educational work. Selections will be based on individual abilities and the importance of the contribution that the applicant's study will make to the improvement and expansion of health services.

The fellowships will cover per diem and travel expenses, and will be mostly limited to travel of 2-4 months. Applications must be received by Feb. 1.

Additional information and application forms may be obtained from Howard M. Kline, secretary, WHO Fellowship Selection Committee, USPHS, Washington 25, D.C.

Richard J. Weisbeck, principal engineering technician for the Erie County Water Authority, died on Jul. 21, 1960, at Buffalo, N.Y., following a two-month illness. Prior to joining the water authority in 1956, he spent 15 years in the sales and service department of Wallace & Tiernan Inc. He had also been chairman of the board of water commissioners and superintendent of the water and sewage department for the town of Alden, N.Y. He was a member of AWWA since 1947.



Employment Information

Classified ads will be accepted only for "Positions Available" or "Position Wanted." Rate: \$1.50 per line (minimum \$5.00), payable before publication. Deadline for ad copy: first of month prior to month of publication desired. To place ad, obtain "Classified Ad Authorization Form" from: Classified Ad Dept., Journal American Water Works Assn., 2 Park Ave., New York 16, N.Y.

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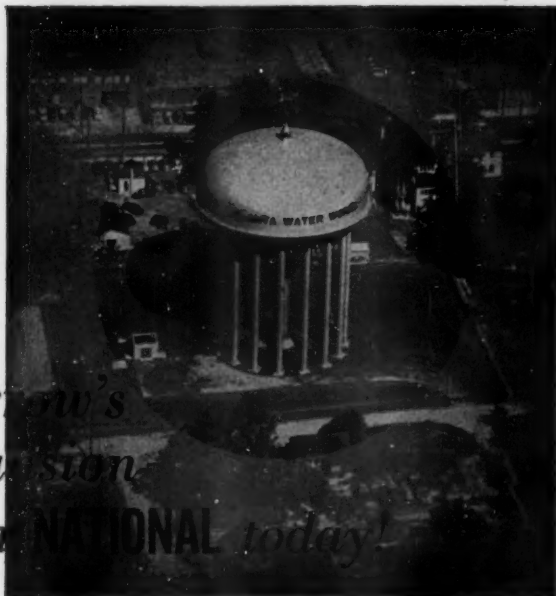
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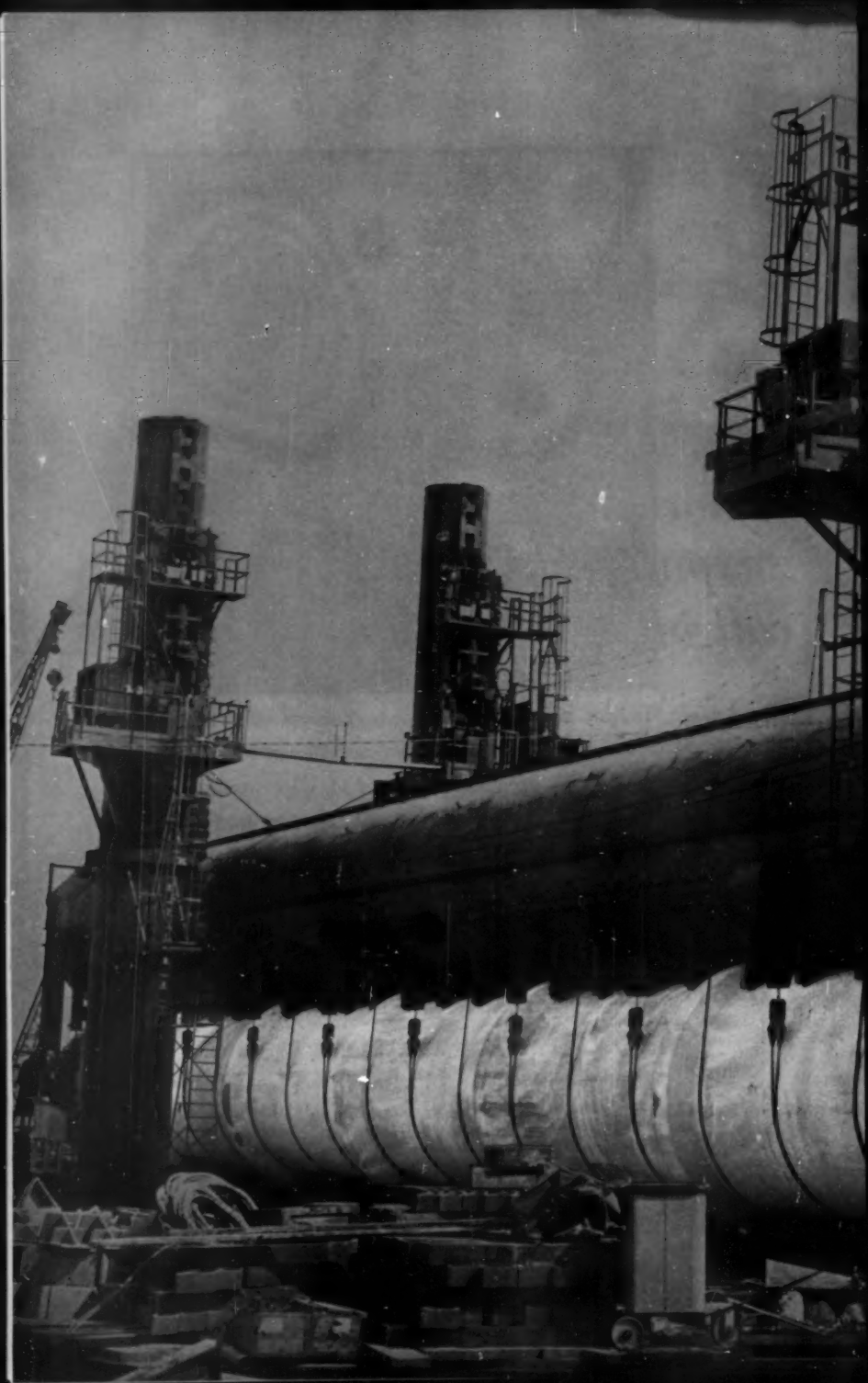
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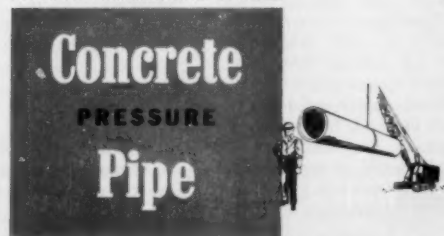
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Key: In the reference to the publication in which the abstracted article appears, 39:473 (May '47) indicates volume 39, page 473, issue dated May 1947. If the pub-

lication is pagged by the issue, 39:5:1 (May '47) indicates volume 39, number 5, page 1, issue dated May 1947. Abbreviations following an abstract indicate that it was taken, by permission, from one of the following periodicals: *BH*—*Bulletin of Hygiene (Great Britain)*; *CA*—*Chemical Abstracts*; *Corr.*—*Corrosion*; *IM*—*Institute of Metals (Great Britain)*; *PHEA*—*Public Health Engineering Abstracts*; *SIW*—*Sewage and Industrial Wastes*; *WPA*—*Water Pollution Abstracts (Great Britain)*.

HEALTH AND HYGIENE

Health and Water Supply. K. MEGAY. *Oesterr. Wasserwirtsch.*, 11:1 ('59). The author surveys the many hygienic problems in the supply of water and the disposal of sewage, discussing separate domestic and industrial water supplies, bacteriologic and chem. requirements of a water supply, the various types and sources of water supply, the disposal of sewage and waste waters and their effects on streams and ground waters, the limitations of artificial treatment of sewage, and the problems of radioactivity in water.—*WPA*

Water Supplies and Public Health in Tropical Africa. C. PEEL. *Sanitarian* (London), 67:9:406 ('59). The problem of water supplies in tropical Africa, especially in N. Nigeria, and the solutions available are described. The outstanding problems are acute shortage of water during the dry season in many areas and the generally low quality of supplies. It is calcd. that on avg. $\frac{1}{4}$ of the African woman's time is spent in obtaining domestic water. In some areas the dry season supply is stored in hundreds of earthenware pots in which *Aedes* breeds happily. Rain-fall wastage results from evapn. or from runoff due to deforestation, overstocking with cattle or shifting cultivation. Water conservation requires integration of many activities and should be organized at the highest governmental level and on a national scale. The development of water supplies has been given a high priority on economic grounds and because they are a basic public health requirement. The installation of a village supply must not be regarded as an end in itself, but as the 1st step in a program of environmental improvement. A piped supply is most popular but the water is often untreated and may become contaminated in the pipes. Under-

ground sources tapped by wells provide a cheap and practicable supply but lifting the water is a problem. A rope and bucket causes contamination and manual pumps and water lifts usually break down. Wind pumps have proved satisfactory in suitable areas in N. Nigeria, but they need skilled servicing. Where they are installed the well can be sealed over and the water pumped to an adjacent service tank. Large earthenware cisterns of about 3-mil gal capacity have been constructed in Nigeria in arid areas in order to conserve rain water during the dry season and have thus far proved satisfactory.—*BH*

Hygienic Supervision of Rural Water Supplies. E. THOFERN. *Desinfect. u. Gesundheitsv.* (Ger.), 50:7:107 ('58). The author describes the unsatisfactory condition of many small water supplies from springs and wells, dealing with protective zones, construction of intakes, pumping plant, reservoirs, and methods of treatment. He recommends cooperation between communities in the appointment of a water controller who would be responsible for supervision of the supplies at all points.—*WPA*

Hygienic Considerations Concerning Connections Between Central and Private Water Supplies. J. WUSTENBERG. *Gas- u. Wasserfach.* (Ger.), 99:945 ('58). From examination of the legal processes arising out of the Hagen epidemic of typhoid in 1955, the author discusses the distribution of responsibility and from the conditions of this outbreak and experience of others, he considers whether private water supplies in towns, especially for food industries, should be permitted and whether existing legislation and control are sufficient to ensure safe separation of supplies of different quality.—*WPA*

(Continued on page 66 P&R)



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(Continued from page 64 P&R)

The Sanitarian in Disaster Situations. E. H. GLEASON, *Military Med.*, 124:354 (May '59). In event of a major disaster, a complete disruption of utility and municipal services, including a complete breakdown of sanitation services, may be anticipated, further complicated by radiological, biological or chemical hazards. The sanitarian is available, motivated, and qualified by virtue of training and experience to advise on matters pertaining to environmental sanitation problems which must be solved immediately include water supply, feeding, food storage, refuse disposal, biologic warfare, disposal of dead, excreta disposal, insect and rodent control, stray animals, air contamination and emergency shelters. "Whether the sanitarian be military, in a health department, or an industrial consultant, his role in passive or civil defense is a role upon which the lives of countless numbers of people may depend. If the time ever comes when disaster strikes, the sanitarian must be prepared to assume his responsibilities to his fellow man."—PHEA

The Hygiene of the Manufacture of Mineral Waters. H. RHEINFURTH & W. ZIMMERMAN. *Arch. Hyg.* (Berlin), 142:609 ('58). Unsatisfactory bacteriologic conditions are frequently found in mineral waters. The author considers that the bacteriological requirements should be higher than those for water supplies. The total contents of a bottle should be free from *Esch. coli* and the total bact. count in gelatine at 22°C should be under 20 and in agar at 37°C under 10 per ml. Most natural mineral waters are bacteria-free but poln. occurs at intakes and from containers and during filling and corking of bottles. The author discusses all these possibilities and methods of control at the works. Sections of various laws dealing with foodstuffs and table waters are described. Health officers have the power to proceed against unclean mineral water factories. The author lists the precautions which must be taken with regard to the intake, the cleaning and filling of bottles, and the bacteriological condition of the finished product. The importance of cooperation between the health authorities and medical and hygienic institutes is emphasized. Hygienic and bacteriologic supervision is a medical duty but large-scale analyses are best carried out by food analysts.—WPA

Water Supply Epidemics—A Historical Survey. H. KRUSE. *Desinfekt. u. Gesundheitsw.* (Ger.), 50:7:101 ('58). The author describes the cause, course, and extent of epidemics caused by water supply in a number of towns and districts. He then discusses briefly the investigation of water epidemics with special reference to the so-called water disease which appears immediately after infection of the water supply and precedes a true typhoid epidemic. It is now generally accepted that infection is not conveyed by drinking pold. water but by contact of the water with some easily pold. foodstuff in suitable conditions of temperature for growth of the bacteria. A water supply may contain typhoid bacteria for years before an epidemic appears but a sudden outbreak may be caused by a combination of unfavorable conditions.—WPA

Epidemiological Study of an Outbreak of Waterborne Dysentery. L. ALTSHULER & D. HERNANDEZ. *Am. J. Public Health*, 49:82 ('59). In a paper presented at a Conference on Preventive Medicine at Washington, D.C., in Feb. '57, a report was given on a waterborne outbreak of dysentery at the US military camp at Panzer Kaserne near Boebingen, Ger. The outbreak was caused by sewage-contaminated water entering the final inspection shaft in the infiltration gallery at Lower Murkenbach water system. Inadequate control of the water supply was criticized and new control measures were enforced.—WPA

Epidemiological Study of a Waterborne Outbreak of Typhoid Fever. A. MASI; R. TIMOTHEE; & R. ARMijo. *Bol. Oficina Sanitaria Panamericana*, 45:4:287 (Oct. '58). This paper describes an outbreak of 23 cases of typhoid fever in the modern town of Ponce, P.R. The pop. is 99,422. The cases occurred within a period of 24 days (Aug. 24 to Sep. 17). All patients, except 3, were under 15 yr of age and all cases were mild. The organism was phage typed and found to be degenerated Vi, which is normally rare in the city. The cases were concd. in 2 central areas, 6 being in one street, and the incidence was 14 times greater in these 2 areas, than in the rest of the city. Apart from water the only article of food or drink taken by the majority of the patients was pasteurized milk of unquestioned

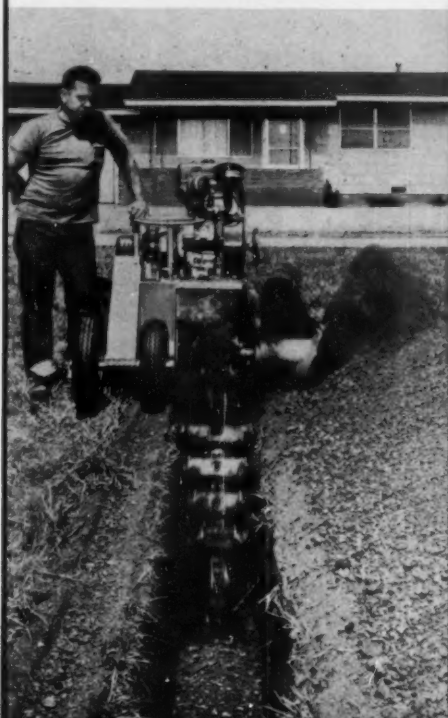
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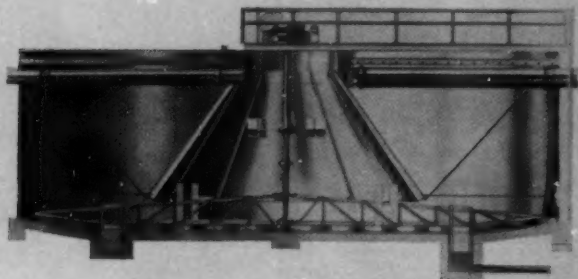
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(Continued from page 66 P&R)

quality. Bact. examination of food handlers was negative. The public water supply is from five deep wells and a reservoir and is chlorinated by an automatic mechanism. The water pressure in the center of the city is low and as, during the dry season, consumers can only draw water at fixed times the water in the distribution pipes is sometimes stagnant for some hours. There are many interconnections within the distribution system. On Aug. 12, the island was struck by a severe hurricane and many refugees crowded into the city, mainly the central area, from the surrounding country. The rare phage, degenerated Vi, had previously been isolated in one of the areas from which they came. The electric supply failed and the normal routine of testing the public water supply was disturbed. The first samples taken after the resumption of testing showed no contamination but on the next day, Aug. 16, the coliform organism content was 16 per 100 ml. A similar result was obtained on Aug. 20 and again at another sampling

point on Aug. 22. Maps show that the distribution of the cases coincided with the area served by the contaminated water. Coliform contamination had been found previously in this section of the water distribution service but it had not been regarded as dangerous. House-to-house inspection in the district showed many sanitary defects and opportunities for contaminating the water supply. For example there was a private well connected directly and without valves with the public service and water pipes adjoining drains were found.—PHEA

Considerations on Waterborne Epidemics of Typhoid Fever in Natural Basins. D. CORNELSON ET AL. *Bul. Ses. St. Inst. Med. Iasi, 1956* (Rum.), p. 1115 ('58). Observations are reported on 2 outbreaks of waterborne typhoid fever in Rumania, one in spring at a village using water from a small river contaminated with waste waters from a community where there had been a typhoid fever epidemic, and the other during autumn

(Continued on page 72 P&R)

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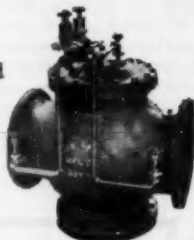


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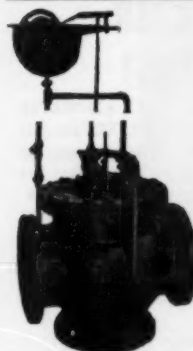
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(Continued from page 70 P&R)

in a community using water from a river contaminated 5 km upstream by waste waters from a town with a high endemic-epidemic potential. The occurrence of the epidemics was found to depend, not only on the gross contamination of the rivers, but also on the reduced flow of the rivers in the period immediately preceding the epidemic, the short distance between the source of poln. and the area consumption of the water, and the reduced self-purification capacity of the rivers during the cold days of spring and autumn.—WPA

Epidemiological and Clinical Features of Tularemia Caused by Waterborne Infection. V. SIL'CHENKO. *J. Microbiol.* (Moscow), 28:788 ('57). Tularemia may be caused by drinking infected water as well as by using it for washing and bathing. Waterborne infection may occur in any season, but is more frequent in autumn and winter when murine rodents migrate into inhabited places. The symptoms of the dis-

ease are described. The best method of preventing tularemia among humans in areas where there are infected waters is by general vaccination.—WPA

Waterborne Epidemics of Leptospirosis. N. BLAGOVESHCHENSKAYA & K. GONCHAROVA. *Hyg. & Sanit.* (Moscow), 24:11:12 ('59). The authors report chem. and biol. investigations, carried out in '55 and '56, on the condition of water reservoirs and rivers in the centers of leptospirosis epidemics. Results are tabulated. Pathogenic leptospirae were isolated from samples of water taken from reservoirs immediately after contamination by the bathing of leptospirae-carrying animals and from samples of river water taken at intervals up to several days after poln. by these animals.—WPA

A Contribution to the Question of the Epidemiology of Anicteric Leptospirosis (Water Fever). B. N. MIKHAILOVSKII. *J. Microbiol., Epidemiol., and Immunobiol.*

(Continued on page 74 P&R)

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(Continued from page 72 P&R)

(USSR), 29:9:1444 ('58). (1) A new endemic focus of water fever has been identified in L'vov Oblast. (2) It has been serologically established that the basic role in the appearance of water fever in this focus has been played by *L. grippotyphosa*. (3) The most probable source of leptospiral infection in the focus has been rodents of the field mouse species. (4) The leading role in the spreading of leptospiral infection in the focus has been played by water. (5) In areas with acid soils, infection with water fever has not been observed. (6) For precise determination of the source of leptospiral infection, extensive laboratory investigation of field rodents and domestic animals is needed.—PHEA

Methemoglobinemia in Infants Caused by Water Containing Nitrate. G. SCHAFER. *Medizinische* (Stuttgart), 115 ('58). The author describes the cause, symptoms, and treatment of methemoglobinemia in infant twins. The cause was the use of water from a polluted well for the preparation of food. The water contained 320 mg/l nitrate. The symptoms of nitrite poisoning, the conversion of nitrate to nitrite by nitrite-forming bacteria in the upper intestinal tract, and the effect of the higher local pH value in infants as compared with adults are discussed.—WPA

Development of Alimentary Methemoglobinemia after the Consumption of Water Containing Nitrates. Z. KNOTEK. *Ceskoslov. Hyg.* (Prague), 5:160 ('60). The authors draw attention to the necessity to study the pathogenesis of alimentary methemoglobinemia and its prevention. They have demonstrated the effect of pH on the reducing capacity of feces, the different bact. capacity of the fecal flora to reduce nitrates in adults and infants and the increased ability to reduce nitrates in some infants. As a result of prelim. data the author assumes that this difference is due to the fact that the reduction is continuous. It is up to future research to confirm this theory and to reveal what causes this continuous reduction. When investigating the effect of $MgSO_4$, corresponding to the degree of different permanent hardness, the author found higher reduction values in vitro. From this he concludes that $MgSO_4$ acts a growth factor.—BH

The Problem of Methemoglobinemia in Infants in Czechoslovakia. J. KELETI. *Ceskoslov. Hyg.* (Prague), 5:152 ('60). Authors present detailed data on the incidence of methemoglobinemia in infants, recorded hitherto in 314 infants and caused by a high nitrate concentration of the drinking water in Czech. Subsequently, the authors discuss the water supply of some areas where the disease is most frequently encountered whereby it was revealed that in the Prague region from 16,008 water specimens investigated 69.9% had a concentration above 35 mg/l and on the Zitny ostrov 77.9% from a tot. of 131 samples investigated. An analysis of infant nutrition revealed that in rural districts the use of artificial feeding is increasing rapidly. From 149 infants examined on the Zitny ostrov only 31.6% were nursed by their mothers for longer than 6 mo. In the present paper the authors evaluate also the effect of the high nitrate concentration of the drinking water on the methemoglobin level in the children examined. From 17 children aged 1-2 yr 13 had a raised methemoglobin blood level without signs of cyanosis or abnormal signs in their physical or mental development. Next the authors describe the clinical course and the results of the treatment of methemoglobinemia of infants in this country. Subsequently they analyze in detail deaths in infants caused by methemoglobinemia and also mention the results of retrospective investigations of infants' deaths from this disease. Finally they evaluate the results of hitherto used measures to prevent methemoglobinemia in infants and recommend that for the preparation of infants' foods suitable sources of drinking water should be sought in rural areas with the assistance of hygiene and epidemiological stations.—BH

Observations on the Epidemiology of Schistosomiasis and Its Control. D. B. McMULLEN & H. W. HARRY. *Bul. Soc. Pathol. Exot.*, 50:901 ('57). To control schistosomiasis, one must consider its prevalence in its vector mollusks, in man, and in its animal reservoirs and its relative importance in its definitive hosts. Reservoir hosts are important for *S. japonicum*, and were recently shown to be important for *S. mansoni* in Brazil and other regions. Proper disposal of human feces or systematic treatment of human cases will not eliminate the

(Continued on page 78 P&R)

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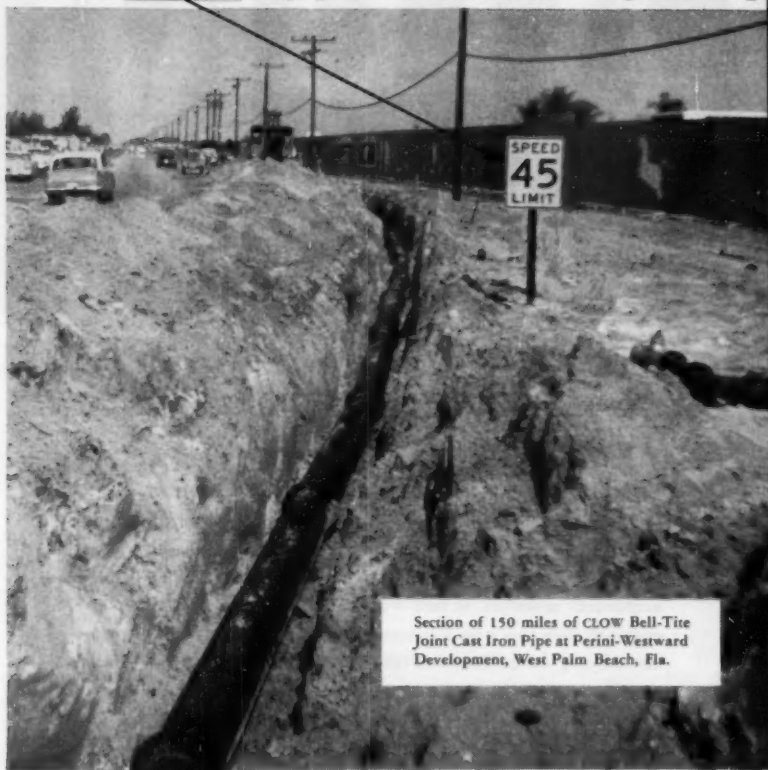
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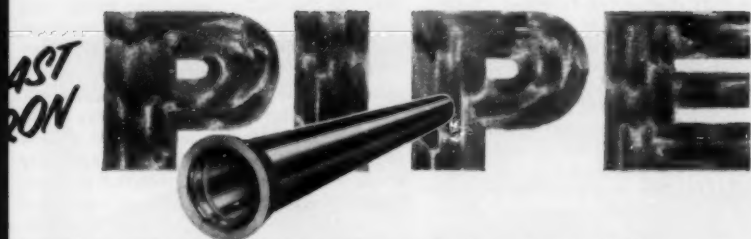
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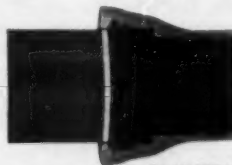
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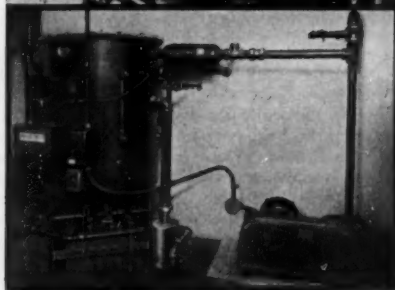
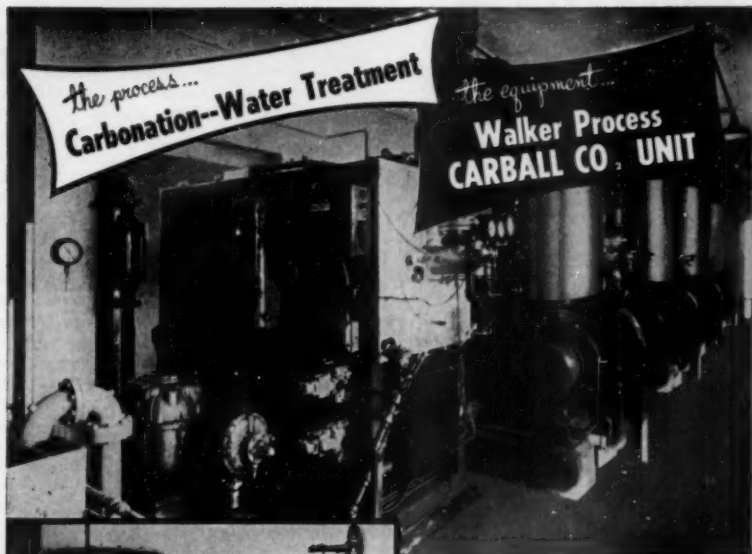
disease if there is an animal reservoir. Elimination of snail vectors by molluskicides or other means will not eliminate the disease if residual infection in man or animals remains to infect the snails after they have become re-established. The following methods are recommended to control schistosomiasis: (1) Prevent pollution of water with human excreta by proper disposal or treatment. (2) Prevent pollution from animal reservoirs, if present, by elimination of rodents or swine dogs and by using horses rather than cattle as draft animals as much as possible. (3) Prevent exposure to cercariae by providing safe water for bathing, washing, and drinking. This can be done by using wells and cisterns and by mechanical or chemical treatment of water from infested sources. (4) Avoid exposure to infested water. (5) Control mollusk vectors by destroying their habitats by drainage and elimination of vegetation; by using pipe or covered channels for irrigation water; by lining the irrigation canals with concrete; by encouraging the natural enemies and diseases of snails; by applying molluskicides (copper sulfate or sodium pentachlorophenate) in relatively heavy concns. at first, repeating the applications at intervals indicated by the conditions, and then continuing low level chemical treatment for a long time; and by drug treatment of positive cases.—PHEA

Biologic Control of *Schistosoma mansoni* in a Puerto Rican Watershed. J. OLIVER-GONZALEZ & F. FERGUSON. *Am. J. Tropical Med. & Hyg.*, 8:56 ('59). The Los Pena community, now emerging from a suburban slum status to a stabilized small housing settlement, has been the object of various schistosomiasis studies since about '33. Trials at stopping transmission among the dense creekside pop. have included Fuadin chemotherapy, health education, improved domestic water supplies, installation of sanitary privies, and a single chem. mollusciciding test. Although those controls were partially effective, the disease remained significantly endemic through '52. Since this time, surveys of the snail intermediate host species, *Australorbis glabratus*, in the main stream showed it to be in precipitous pop. decline faced with a phenomenally large colony of an ampullarid snail, *Marisa cornuarietis*, first seen in the stream in '52. During

Mar. '56 this highly competitive species was successfully transplanted to all upstream stations in the watershed, and during a period of one and a half years has almost eliminated the host species in these areas. Two recent successive annual surveys of the same creekside families indicate that no new infections are appearing in the statistically sensitive preschool age group. The community prevalence of all ages and both sexes has leveled off at a rate of about 6 or 7%. This first instance of the biological control of schistosomiasis transmission should be repeated in other endemic environments capable of supporting the remarkable snail *Marisa*. This field test is a step toward the solution of certain control problems in Puerto Rican schistosomiasis on a permanent and economically feasible basis.—PHEA

The Removal of Dangerous Concentrations of Nitrate from Water. I. DELIUS. *Gesundh.-Ing. (Ger.)*, 80:6:181 ('59). Reference to the methemoglobinemia which may occur in artificially fed infants whose feeds have been made up with water of high nitrate concn. introduces this account of expts. undertaken to study methods for the removal of the nitrate ion. Chem. reduction of nitrate by metals—iron, aluminum, or magnesium—was found not to be satisfactory. Elimination of nitrate by the addition of cultures of algae has been investigated by some workers but found to be impracticable. Most of this paper is devoted to an account of expts. on the use of the method of ion exchange. The material used was a strongly basic anion exchange product. Sulfate, chloride, or carbonate anions were introduced by treatment of ES with the sodium salts or these anions followed by thorough washing. Water from the Berlin supply to which potassium nitrate was added to bring the concn. of NO₃ to 100 ppm was then filtered through each of the 3 anion-exchange materials. The nitrate ion was removed by all 3 but the efficiency of exchange was greater by the chloride ion and the bicarbonate ion than by the sulfate ion material. It was found, for example, in filtration expts. with the chloride-ion material that 50 g of this removed all the nitrate from more than 24 liters of water. Full numerical details of the results are set forth in tables with a complete discussion.—BH

(Continued on page 80 P&R)



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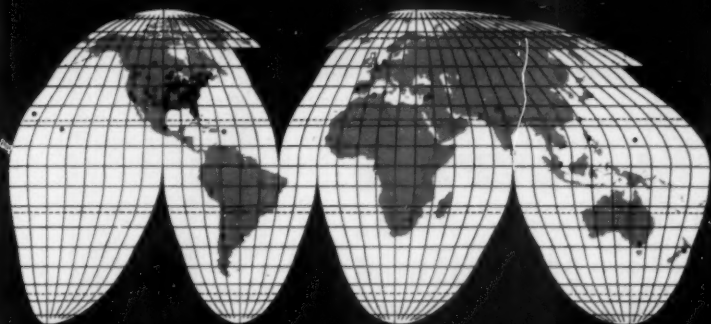
(Continued from page 78 P&R)

An Outbreak of Salmonella Food Poisoning in Osimo, Italy. A. BALICE. *Igiene e San. Pubblica* (Rome), 14:612 ('58). Osimo is a commune near Ancona close to the Adriatic coast of Italy and the author describes an outbreak of food poisoning in which 83 persons living in a farm colony were affected, 12 were children between 6 and 12 yr of age; at least 47 had to be admitted to hospital, and 5 died. Investigations proved that the infecting vehicle was poultry meat eaten at a premarital feast (at which the bride's dowry is demonstrated to those interested). The meal was eaten at 1 PM and the usual symptoms of vomiting, diarrhea, and fever started 9-30 hr later. Circulatory collapse was a prominent symptom in several patients. 2 children who did not eat the meat but ate other portions of the feast were unaffected, and one person who was not at the meal but who drank out of the same glass as one of the patients also became ill. Necropsy of the 5 fatal cases showed gastroenteritis with hemorrhagic glomerulonephritis in 2. For the remainder the illness lasted 1-12 days, and the presumed causal organism was isolated from the feces for at least 30 days after. After plucking, drawing, and cleaning, the poultry were washed in water from an old well not used for drinking, and then were put aside for more than 12 hr in a fairly high ambient temp. before being roasted or boiled. An organism belonging to the *Salmonella* group with B somatic and e,n,x, flagellar antigens was isolated from the well water, and from the feces, intestine, spleen, and peritoneal fluid of the fatal cases. Blood culture from some of the other patients was negative. Ducks and geese from the farm yard killed subsequently proved to be healthy carriers of the same organism. The author considers that these birds infected the well which structurally was in a very poor state, and that the subsequent infection of the cleaned carcasses was favored by the long time they were kept before cooking.—BH

The Danger of Lead Poisoning by Addition of Condensed Phosphate to Scale-Forming Tap Waters. R. BUYDENS. *Bul. Acad. Roy. Med. Belg.*, 22:293 ('57). The author describes investigations into the solution of lead in water treated with phosphate. He concludes that addition of phosphate to the whole supply is not advisable, as no pro-

(Continued on page 82 P&R)

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(Continued from page 80 P&R)

fective film can form and lead will be dissolved, and that the domestic use of phosphate containers through which the water is passed should not be permitted where there are lead pipes.—WPA

Chronic Toxicity Studies—II. Hexavalent and Trivalent Chromium Administered in Drinking Water to Rats. R. D. MAC-KENZIE ET AL. *A.M.A. Arch. Ind. Health*, 18:232 ('58). 5 groups of rats were given water containing between 0.45 and 11 ppm of chromium as chromate ion for 1 yr. No differences were observed between these groups and the controls as to water intake, food consumption, or weight gain; nor did an analysis of blood at monthly intervals or examination of tissues at 6 mo or 1 yr show any significant differences between any of the groups given chromium and the control group. Kidney, liver, and femur were analyzed for chromium at the end of 6 mo, and these plus spleen were analyzed at the end of 1 yr. There was an abrupt rise in tissue chromium concn. when rats ingested water having concns. above 5 ppm chromate ion. 2 groups of rats were given water containing 25 ppm chromium as hexavalent and trivalent chromium, respectively, for 1 yr. No toxic symptoms were observed in either group; however, tissue concns. were approx. 9 times higher in the group given hexavalent chromium. This suggests that chromate ion is absorbed to a much greater extent than trivalent chromium at this level.—PHEA

Significance of Ecological Studies of Wild-Animal Reservoirs of Zoonoses. M. ANDUSSALAM. *Bul. World Health Organization*, 21:2:179 ('59). Lack of information on the ecology of wild-animal reservoirs of zoonoses has led to failure in control and eradication of these diseases. In some cases, ecologic studies on the characteristics of the terrain, climate, vegetation, topography, and other environmental factors (Pavlovsky's "landscape epidemiology") have revealed the existence of enzootic foci of certain diseases. It is evident that there is a great need of extending the ecologic approach to the study and control of zoonoses in wild-animal reservoirs, especially when it concerns vector-transmitted diseases. Aside from the interest in control and eradication, ecologic research may reveal new zoonoses and may

contribute basic knowledge to other biological disciplines. It is also possible to discover new animals which might be used for laboratory expts. in the study of disease. The principal characteristics of zoonoses worth studying include identification of all animals in the immediate environment including their parasites; life history studies; statistics and population dynamics of host and vector; biotic, climatic and physical characteristics of the environment; the animal's interrelationship to enemies, predators and parasites; food supply; examination of host and parasite tissues for the presence of pathogenic organisms and of morbid changes and antibodies. Other ecologic studies of the vector such as the ability of the infective agent to multiply in its tissues, or, in the case of certain arthropods, the development of behavioristic resistance to insecticides, should be fruitful. The organization of ecologic surveys of enzootic foci in varied environments necessitates the cooperation of scientists representing many fields of endeavor. The author concludes with a list of the principal zoonoses with wild-animal reservoirs.—PHEA

Effect of Synthetic Fatty Acids on the Sanitary Regimen of a Reservoir. N. V. GRIN. *Hyg. & Sanit. (Moscow)*, 23:12:13 ('58). The effect of fractions of synthetic fatty acids of various molecular structures on reservoirs was studied, with respect to the effect on BOD, ammonification and nitrification, and saprophytic organisms, and the effect of several groups on odors. It was found that the molecular size is inversely proportional to the intensity of its effect. When the water contained a group of fatty acids, they had a combined effect on odor and 5-day BOD of the water.—WPA

FLUORIDATION

Fluoridation and Dental Caries. G. LAPAGE. *Nature (London)*, 182:1351 ('58). A recent report of the Swedish Royal Medical Board states that fluoridation of water supplies for the control of dental caries does not involve any demonstrable health hazards. It should, however, be carried out under strict control, and care should be taken that people drinking fluoridated water are not exposed to fluorides derived from other sources such as foods or industrial gases.

(Continued on page 84 P&R)

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(Continued from page 82 P&R)

A concn. of about 1 ppm fluoride will reduce the incidence of dental caries by about 50% in children who have consumed this water all their lives. Fluoride also seems to protect the teeth of middle-aged people. The cost of fluoridating the water supplies for some Swedish cities is estimated.—*WPA*

Fluoridation in North Dakota, Present and Future. J. K. PETERSON. *Off. Bul. N. Dak. Wtr. Wks. Conf.*, 27:3:13 ('59). The historical events leading to the fluoridation of water supplies and its beneficial effects in the subsequent prevention of dental caries are outlined briefly. A table listing 14 towns in N.D. which fluoridate their water supplies, their fluoridating agent and method of fluoridation, and the cost at each plant, is given. The cost of fluoridation is considered worth while, both for the reduction in dental caries and for the reduction in money spent on dental care. This is illustrated by comparisons between 2 towns in Idaho and Wisconsin. A number of towns

in N.D. have an excess of fluoride in their water supply, which causes fluorosis and teeth mottling; the prohibitive cost for the removal of fluorine prevents this being carried out.—*WPA*

Fluorine as Dental Caries Prevention Factor. R. GABOVICH & P. MAISTRUK. *Hyg. & Sanit. (Moscow)*, 24:7:21 ('59). A survey was carried out to investigate the effect of fluorine concn. in drinking water on the incidence of dental caries in the Ukraine. 80,000 persons drinking water containing between 0.03 and 4.2 ppm fluorine were examined, and it was found that the incidence of dental caries was very high in all age groups of the population where the fluorine concn. in the drinking water was low, especially below 0.5 ppm. Similar investigations repeated after 10 yr revealed that where the fluorine concn. in drinking water was reduced from 2-2.5 to 1.8-2.0 ppm, the incidence of fluorosis decreased from 52.8% in '48 to 27.3% in '58; where the fluorine concn.

(Continued on page 86 P&R)

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(Continued from page 84 P&R)

was less than 0.3 ppm, the incidence of dental caries had not decreased in spite of great improvements in living conditions and nutrition during the last 10 yr, and in settlements where fluorine concn. in water was sufficiently high, the dental caries rate was as low as in '48. It is recommended that investigations be carried out using between 0.9 and 1 ppm fluorine in winter and between 0.7 and 0.8 ppm in summer to determine the min. dose of fluoride required for fluoridation of water. The possibility of fluoridation of table salt, milk, and other food products is also to be studied.—WPA

Prevention of Dental Caries by the Fluoridation of Public Water Supplies.

W. ALCOCK. *J. British Waterworks Assn.*, 41:653 ('59). In a paper presented at a meeting of the British Waterworks Assn., London, Nov. '59, the problem of the widespread occurrence of dental decay, due to too much starch and sugar in present-day diets, was discussed, and early research on the effect of small concns. of fluoride in drinking water on the health of teeth was outlined. A brief description of work carried out in the US and Canada establishing the beneficial effects of fluoridation and expts. carried out in 6 study areas in the US to determine the effects of controlled fluoridation on children's teeth was given. Results indicated an extensive reduction in the occurrence of dental caries when a concn. of 1-1.2 ppm fluoride was added to the water supplies. Fluoridation of water supplies in 3 controlled study areas in Gr.Br. (namely Anglesey, Kilmar-nock, and Watford) is now in progress, to provide information on its effect on dental caries in this country. Results obtained so far are favorable, and if these continue to be satisfactory, the introduction of fluoridation of all water supplies not having a natural fluoride content is recommended. The advantages of fluoridation were discussed. In notes appended to this paper on the water treatment plant at Watford, a general description of the fluoridation equipment at the Watford Fields and the Grove pumping stations, and its operation, was given. The former uses sodium fluoride and the latter sodium silicofluoride, to provide a concn. of 1 ppm fluoride in the water. Costs were given. In discussion, arguments for and against fluoridation were put forward, and further information on the effects of fluoridation was presented.—WPA

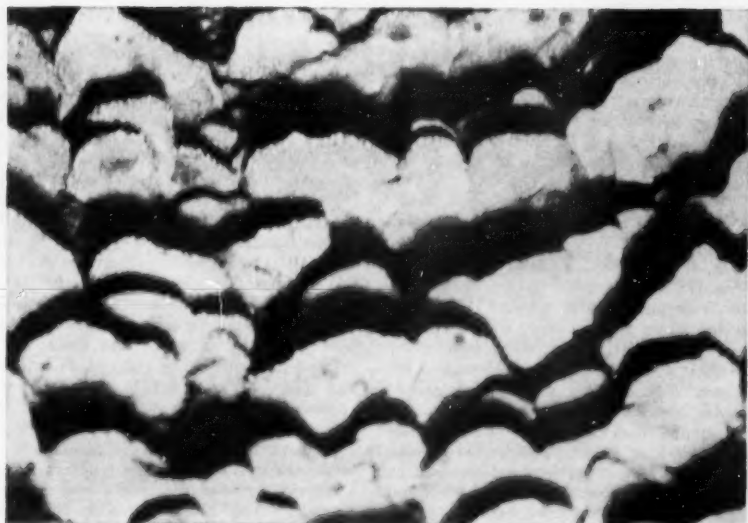
Experience in Maintaining Constant Fluoride Concentrations.

G. SPITZ; F. TAYLOR; & W. HARRIS. *Am. J. Public Health*, 48: 1651 ('58). To determine the constancy and adequacy of the fluoride concn. in water supplies, water samples were collected at frequent intervals from several points in the treatment and distribution systems of Grand Rapids, Mich., and of two other towns in eastern US. Results of analyses are presented in tables and graphs, and are discussed in relation to the importance of maintaining the required concn. of fluoride for the prevention of dental caries. Fluoridation was found to be adequate and fairly constant at Grand Rapids, but it showed some deviation in the other two towns, and it was recommended that these deviations should be corrected as soon as possible.—WPA

Natural Fluoridation and Mottling of Teeth in Lincolnshire.

J. CLARKE & J. MANN. *Brit. Dental J.*, 108:5:181 ('60). This paper contains an interesting description of the geology of Lincolnshire in relation to the underground water supplies and the differences in the concn. of fluoride in various areas. From these observations it is concluded that the cause of natural fluoridation is a base exchange process, the chemical agent involved in natural fluoridation being sodium fluoride. There is also an account of the results of a dental survey of a sample of 56 children aged 9-15 yr, in an area where the public water supply contains 2.5 ppm fluorine, to classify the extent of mottling present into the grades "normal," "questionable," "very mild," "mild," "moderate," "moderately severe," and "severe." The results were that only 2 children had mottling as pronounced as "moderate," 16 were normal and in 6 there were idiopathic spots. Some photographs of the anterior teeth of some of the children were compared with similar pictures taken 5 years earlier to ascertain whether changes in the degree of mottling had occurred. It is stated that dental inspections in the area where the fluoride concn. of the public water supply has been 2.5 ppm for many years have never disclosed any disfigurement of teeth due to mottling. Only in some children who consumed water from wells containing 4.0 ppm did white and brown stains appear which were obvious to the ordinary observer. It

(Continued on page 88 P&R)



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(Continued from page 86 P&R)

is concluded that water naturally fluoridated at 2.5 ppm brings about a 60% reduction in the incidence of caries and that the "mottling effects do not detract from the aesthetic appearance of the teeth." No adverse effects in the general health of the pop. could be discovered.—BH

A System for Fluoridating Individual Water Supplies. F. J. MAIER. *Am. J. Public Health*, 48:717 ('58). Only $\frac{1}{3}$ of the people of the US will be able to get fluoridated water from public water supplies. The rest, who depend on private, home water systems, cannot at present obtain fluoridated water from their sources of supply. However, for the past 2 years the water in 4 suburban Md. homes has been treated to provide the optimum fluoride concn. The installations consist of a timer controlled by the well pump motor starter, a solenoid or hydraulically actuated soln. microfeeder, and a 1-gal reservoir for the fluoride soln., all housed in a locked box. Experience with these feeders indicates that a consistent fluoride level can be maintained in home water systems if satisfactory maintenance and adjustment schedules are maintained. For this reason it is advocated that servicing organizations similar to those now providing chlorinated or softened water to householders should maintain the fluoridation equipment and be responsible for the entire installation. Cost to the homeowner is estimated to be about \$3 per month.—PHEA

The Physiologic and Hygienic Aspects of Absorption of Inorganic Fluorides. *Arch. Ind. Health*, 21:303 ('60). This is a valuable and interesting symposium of 18 papers and summaries of papers on recent scientific work on many aspects of the subject. The first communication is a summary of a report on dental surveys made in 3 districts in the US in which the communal water supplies have been fluoridated since 1945. The promotion of dental health in this time is clear. Topical application of fluoride, diet supplements of fluoride and fluoride dentifrices are also discussed shortly. There are short accounts of analytical procedures in which the Megregian reagent (Zirconyl-eriochrome-cyanine R) is used which give satisfactory results with 1-5 μ g of fluoride in certain materials. Some of the results of analysis of air, food, and water are given which indicate the omnipresence of fluoride and the

great variations in its concn. On the subject of the metabolism of fluorides in man there is an editorial summary of 4 papers published elsewhere. Expts. in man have shown that: (1) the absorption by the alimentary tract of fluoride ingested in the form of sodium fluoride, calcium fluoride, cryolite, and rock phosphate is remarkably effective; (2) roughly half of the absorbed simple fluoride salts is excreted by the urine and half is retained mainly in the skeleton; (3) sodium fluoroborate, while readily absorbed (98%), is completely excreted by the urine and is not retained in the body; (4) when the rate of intake and absorption of fluoride is appreciably reduced the fluoride that has been retained in the skeleton is released slowly over a long period; (5) excretion of fluoride in sweat is a highly significant feature of its metabolism; and (6) in general the human body tends to achieve a balance between the intake and output of fluoride. The section of this symposium which deals with the effects of the absorption of fluoride contains 8 summarized reports on investigations on different facets of the subject. (a) In a 10-yr investigation of more than 100 persons who for over 30 years had used water containing 8 ppm of fluoride it was found that "no clinically significant, adverse, physiological, or functional effects resulted from the absorption of the fluoride, with the exception of some dental fluorosis." (b) study by x-ray of 546 persons aged 30-70 yr in Framingham, Mass., where the water contains less than 0.1 ppm fluoride, provided evidence that an insufficient amount of fluoride in adult bone is disadvantageous. Among various points in this evidence was the finding of 229 cases of osteoporosis (c) several summaries of papers containing information on the findings of histological and chemical examination of human bone and soft tissues under variable conditions of fluoride ingestion. Other papers deal with the important subject of fluoride intoxication in animals. In one of these papers there is a description, with references, of the results of expts. on induced clinical fluoride intoxication in animals in which there appears to be a species tolerance to the element. The tolerance in a given species is "modified downwards by factors of stress, such as infection, increased energy demands, shifts in certain dietary factors, and faulty nutrition." Another communication reports on the changes following prolonged feeding of fluoride to

(Continued on page 90 P&R)

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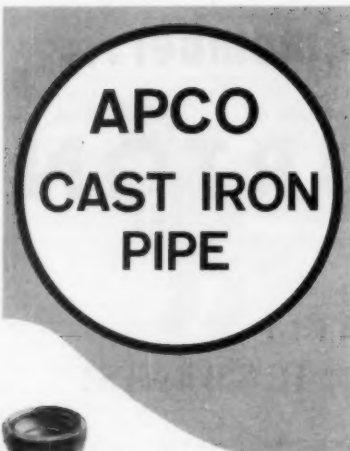
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(Continued from page 88 P&R)

cattle and sheep from which it is concluded that dental fluorosis is an excellent clinical sign in the diagnosis of fluorosis in cattle and that definite radiological and histopathological changes have been observed in bone of cattle that contains 4,000 ppm fluoride. Studies of hepatic function in cattle by serum bilirubin concn., sulfobromophthalein clearance and thymol turbidity tests showed that there was no indication of gross effect on liver function of ingestion of fluoride in concn. of 100 ppm of the diet for a period of 5.5 yr. A short final paper contains notes on sources of fluoride, routes of entrance into the body, fluoride effects and a list of important, unanswered questions.—BH

Dental Fluorosis in Cape Verde—Study of an Endemic Focus in St. Vincent. M. DE MEIRA & H. DE SOUSA. *Anais Inst. Med. Trop.* (Lisbon), 14:3/4:429 ('57). In one small village where the drinking water contains 3.3-4.55 ppm fluorine, 73% of the inhabitants showed dental fluorosis. Other villages had less fluorine in the water and less fluorosis. The incidence of dental caries was high in the village with a high incidence of fluorosis. There is evidently an optimal concentration of fluorine (0.5-1.0 ppm) to confer protection against caries.—BH

Topical Applications of Sodium Fluoride and Stannous Fluoride. G. NEVITT; D. WITTER; & W. BOWMAN. *Pub. Health Repts.*, 73:9:847 ('58). There have been reports which suggest that stannous fluoride is more effective than sodium fluoride in reducing the solubility of powdered enamel, and some further reports suggest that stannous fluoride applied topically to teeth in the mouth is more effective in reducing dental caries than is sodium fluoride similarly applied. In '55-'57 the authors carried out an investigation in 2 areas in Oregon—children of 9-14 years of age having a 2% stannous fluoride solution applied to the teeth in 2 mouth quadrants and a similar number of children (almost 300) having an application of 2% sodium fluoride. The untreated quadrants served as controls. 16 mo after the initial examination the incidence of dental caries in permanent noncarious teeth was significantly lower in mouth quadrants treated with either stannous fluoride or sodium fluoride than in untreated quadrants. The findings relative to the comparative

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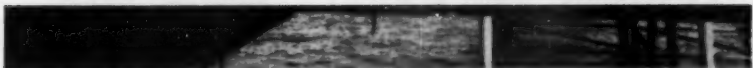
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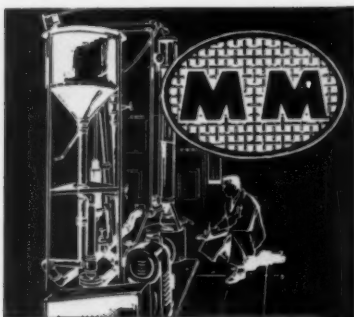
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(Continued from page 90 P&R)

effectiveness of stannous fluoride and sodium fluoride were inconclusive.—BH

RADIOACTIVITY

Disposal of Radioactive Liquids From Nuclear-Powered Ships. J. M. SMITH. *Sew. & Ind. Wastes*, 31:1323 ('59). The author discusses the problem of the disposal of radioactive waste waters from nuclear-powered ships, dealing specifically with those using water-cooled and moderated reactor systems. Solid wastes will consist of resins from the demineralizers in the primary water system and from the purification of condensate. Liquid radioactive wastes will be mainly regenerating solution from the condensate demineralizers together with some leakage of high-purity water. From a consideration of the present no. of large merchant vessels in the world, it is thought that there will not be more than 100 nuclear ships, and even with naval vessels the total number is unlikely to be more than 300, of which only a very few are likely to be in one harbor at the same time. A study group, set up in 1958 by the National Academy of Sciences National Research Council has proposed criteria for the disposal of wastes from nuclear ships in harbors and coastal waters, on the outer continental shelf, and in the open sea. It is recommended that when possible disposal should be carried out far from land, away from other ships, and not in fishing areas. It is concluded that, provided care is taken, there should be no difficulty in disposing of these wastes at sea.—WPA

Concentration, Treatment, and Storage of Radioactive Wastes. V. I. SPITSYN. *J. Appl. Chem.* (London), 9:315 ('59). After reviewing methods used in the treatment and storage of radioactive waste waters in France, Belgium, Gr.Br., and the US, the author describes processes used in Russia. In one process, dilute radioactive waste waters of low activity are treated with basic ferric sulfate and sodium hydroxide causing copptn. of zirconium-95, niobium-95, rare-earth isotopes, some ruthenium-103 and ruthenium-106, and a little strontium-89 and strontium-90; and the ppt is removed by quartz filtration and buried. The strontium, ruthenium, and cesium-137 remaining in the filtrate are almost completely removed by passage through cation- and anion-exchange columns.

(Continued on page 94 P&R)

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(Continued from page 92 P&R)

The waste waters from a radiobiological research lab. are treated by preliminary aeration and sedimentation, coagulation with ferric hydroxide, and filtration through sand or anthracite.—WPA

Liquid Waste Disposal Methods at Bettis Hot Laboratory. W. BRYANT & L. BEHR. *Proc. 7th Hot Lab. & Equip. Conf.*, p. 397 ('59). The treatment and disposal of radioactive waste waters from the Bettis Hot Lab. of the Westinghouse Electric Corp., Pittsburgh, Pa., are described. The activity of the waste waters must be reduced from over 100 mr/hr/gal to 20 mr/hr/gal before the effluent can be discharged to the main plant processing system. The contaminants, which consist of particulate matter and soluble ions, must then be prepared for burial at sea. Methods of reducing treatment costs have been considered.—WPA

A Method for the Solidification of Radioactive Aqueous Liquid Wastes. J. KAUFMAN & N. GARDEN. *Nuclear Sci. Abstr.*

13:2326 ('59). A method is described for the solidification of radioactive waste waters, by adding portland cement, mixed with expanded vermiculite until all the liquid is absorbed. Detailed procedure for applying the method to various waste waters, is outlined.—WPA

Preliminary Tests on Clay Sinters to Retain Reactor Wastes. J. M. KERR. *Bul. Am. Ceram. Soc.*, 38:374 ('59). The fixation of radioactive isotopes by clays and other fine-grained naturally-occurring materials was studied, and the min. amt. of clay required to retain the activity was determined. The best results were obtained with 100 g of shale with fluxes, 50 g of phosphate slime, or 50 g of a processed attapulgite clay, each of which would retain 250 ml of a simulated waste soln. containing aluminium nitrate, nitric and sulfuric acids, mixed fission products, and carriers. Some expts. with relatively high activities in the waste soln. indicated that increased firing temps.

(Continued on page 98 P&R)

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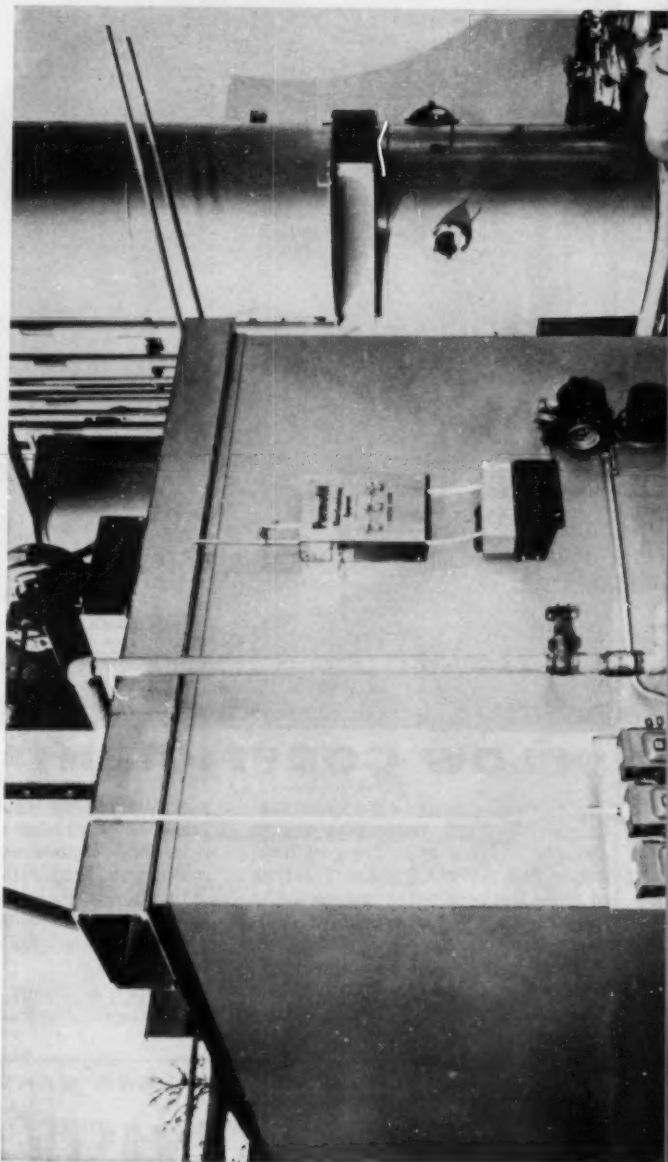
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(Continued from page 94 P&R)

would be necessary to retain the higher activity.—WPA

Winfrith Heath Atomic Energy Establishment. *Fluid Handling* (Gr.Br.), 115:219 ('59). The Winfrith Heath atomic energy establishment, at present under construction, is described. All radioactive waste waters will be treated and discharged to the sea, permissible limits for their discharge having been determined from the results of an extensive research program carried out during the latter half of '56. The radioactive waste waters will drain into delay tanks in which they will be held until the level of activity is measured. Depending on the activity level, the liquid will be pumped to one of three 200,000-gal active sea disposal tanks for batching and final analysis, or diverted to a treatment plant in which the activity level in the liquid will be reduced by chemical treatment. The nonradioactive effluent will be collected in two 200,000-gal nonactive disposal tanks. Both types of effluent will be discharged to sea through double pipes, the radioactive wastes flowing through the inner pipe and the nonactive wastes through the outer annulus; any leak from the inner pipe will be detected by automatic monitors on the outer stream. Storm water from the eastern section of the site will be drained into the river Win, and that from the western section into a watercourse flowing into the river Frome. Sewage from the western active area will be treated at a plant at the western end of the central area and the effluent will be discharged to sea. Sewage from the nonactive eastern area will be treated temporarily at the Authority's plant, but will eventually be sent to a new works to be built by the Wareham and Purbeck RDC to treat the inactive waste waters. There are two sources of water supply available; the supply being used at present is drawn from a well at Briants Puddle on the river Trent, but there are wells located at West Stafford which are capable of supplying the estimated future requirements of the establishment.—WPA

Separation of Strontium-90 From Solutions. K. LIESER & W. HILD. *Naturwissenschaften* (Ger.), 46:599 ('59). The results are described of expts. on isotope or ion exchange between strontium-90 and solid sulfates of calcium and barium. Low concns. of strontium-90 in aqueous solns. can be con-

siderably reduced within a few minutes by shaking with finely divided solid barium sulfate. With longer treatment, because of recrystallization of the barium sulfate, still further strontium ions are removed. It is to be expected that repetition of the process, for example, by using separating columns filled with solid barium sulfate, would give still better separation of strontium-90 from aqueous soln.—WPA

Ultimate Disposal of Liquid Radioactive Wastes. L. P. HATCH. *Nuclear Sci. Abstr.*, 13:3083 ('59). The author stresses the importance of segregating low-level, intermediate, and high-level radioactive waste waters, so that full advantage may be taken of the capacity of the soil to remove radioactive materials from large volumes of low-level wastes, while the more elaborate and expensive disposal methods, such as permanent storage in salt cavities and fixation in stable media, are reserved for high-level wastes.—WPA

The Accumulation of Radioelements in Certain Groups of Water Organisms. D. IL'IN; Y. MOSKALEV; & A. PETROVA. *Atomic Energy* (USSR), 5:171 ('58). Studies of the accumulation of radioactive elements by living organisms in water have shown selective uptake of phosphorus-32, strontium-89, strontium-90, cesium-137, and sodium-24 by plankton and fish. The concn. of phosphorus-32 was 3-4 times higher in fish, plankton, and deep-sea organisms than in water, and the concns. of strontium-89, strontium-90, and cesium-137 were 2-3 times higher.—WPA

Certain Data Concerning Cumulation of Polonium-210 by Water Organisms. V. Z. AGRANAT. *Med. Radiol.* (USSR), 3:1:65 ('58). It was found that there was marked accumulation of polonium-210 in the bodies of fish in reservoirs where the concn. of polonium was much less than the max. permissible concn. of 5×10^{-11} c. It is suggested that the max. permissible concn. of polonium in water should be reduced.—WPA

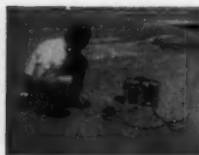
Transmission of Radiostrontium by Nutritional Chain From Open Water Reservoirs Into the Human Organism. A. MARET; M. SAUROV; & G. LEBEDEV. *Med. Radiol.* (USSR), 3:1:69 ('58). Aquatic organisms such as plankton, aquatic plants,

(Continued on page 100 P&R)

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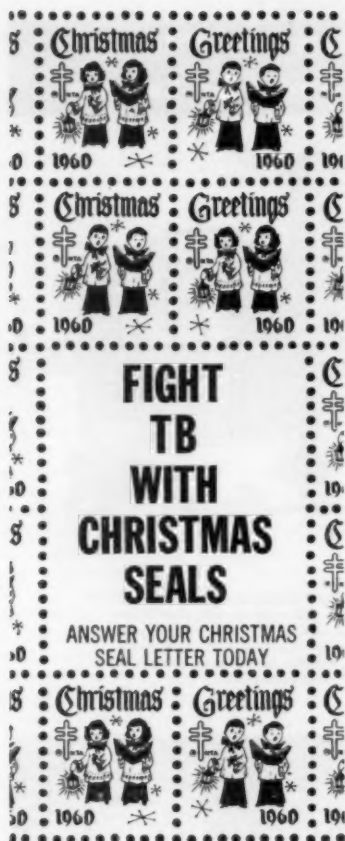
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(Continued from page 98 P&R)

mollusks and fish, as well as domestic ducks, accumulate significant amts. of radioactive strontium in their tissues from the water, and the isotope may pass into the human organism when fish and poultry are eaten. In view of this, it is suggested that the max. permissible concns. of radioactive strontium in reservoirs should be revised.—WPA

Radioactive Matter in Water and Waste Water. B. DIETERICH. *Vom Wasser* (Ger.), 25:163 ('58). After a brief account of the dangers of radioactive poln. of water, the author describes the methods and calculations used in determining max. permissible concns. and gives figures obtained for some isotopes affecting different organs and parts of the body. The sources, types, and amounts of radioactive wastes are then described and an account is given of methods of treatment and their efficiency, including storage, evaporation, chem. treatment, ion exchange, and absorption in clay, and disposal at sea or in other places, and a combination of processes used at the reactor plant.—WPA

The Hygienic Characteristics of Natural Radioactivity of the Soil. V. N. GUSKOVA & A. N. BRAGINA. *Hyg. & Sanit.* (Moscow), 23:10:32 ('58). Results of investigations on the natural radioactivity of soil are described. Potassium-40 is responsible for 20-76% of the radioactivity of the soil. After burying solid radioactive wastes the surrounding territory in some instances remains contaminated and it is therefore necessary to keep the burial places under strict control and to examine the soil occasionally for radioactivity.—WPA

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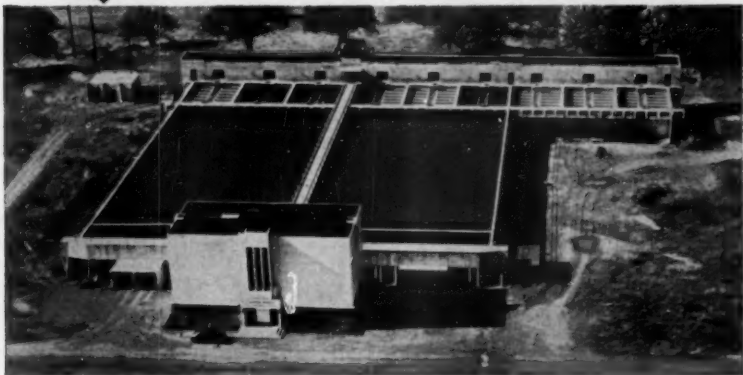
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Correspondence



One Wrong Makes Three Write

To the Editor:

Re the letter by E. A. Sigworth on female statistics (August P&R, p. 54), may I suggest that he check his calculations. My slide rule reads closer to 29 per cent. The millennium is closer than he thinks.

T. E. LARSON

Urbana, Ill.
Aug. 25, 1960

* * *

To the Editor:

... I disagree with Mr. Sigworth's arithmetic as well as his conclusions. The following table, I believe, is arithmetically correct:

	Men	Ladies	Per- centage
1960 (Bal Harbour)	2157	879	29.0
1959 (San Francisco)	2426	785	24.4
1958 (Dallas)	2337	687	22.7

From a statistical standpoint it is not proper to draw conclusions from these years alone. Let's go back to the June

Willing Water and pick up the preceding seven years and then see where we are:

	Men	Ladies	Per- centage
1957 (Atlantic City)	2398	669	21.8
1956 (St. Louis)	2026	510	20.1
1955 (Chicago)	2075	512	19.8
1954 (Seattle)	1536	527	25.5
1953 (Grand Rapids)	1532	365	19.2
1952 (Kansas City)	1600	386	19.4
1951 (Miami)	1415	491	25.8

If we had looked at the attendance after 1953 we would have concluded that by 1964 or 1965 there wouldn't be any ladies attending the annual AWWA Meetings. Horrors!

On the basis of the 10 years' data given above, it is reasonable to predict that the lady registrants probably will never exceed 35 per cent or so—provided, of course, that significant population changes do not occur.

ROBERT C. SISCO

Oakland, Calif.
Sep. 7, 1960

* * *

To the Editor:

In reference to your comment at the bottom of Mr. Sigworth's letter, are you sure you weren't more interested in the statistics of females than in female statistics?

G. R. SCOTT

Kansas City, Mo.
Sep. 7, 1960

To friend Sigworth, the Correspondence column gives thanks for a 300 per cent bonus.—Ed.

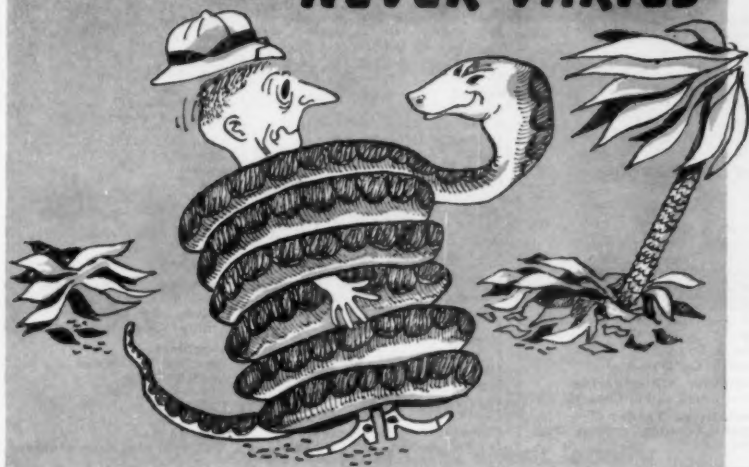
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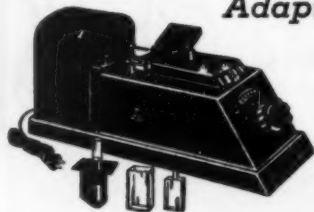
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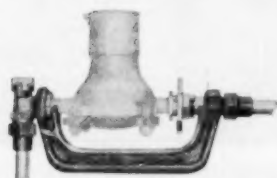
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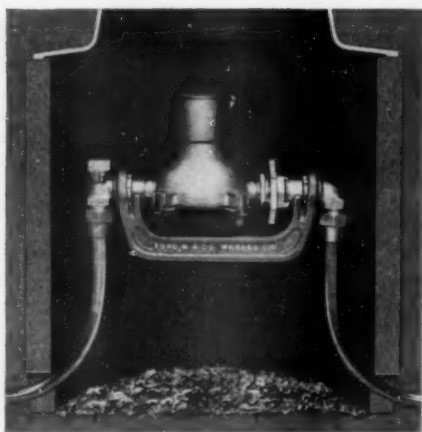
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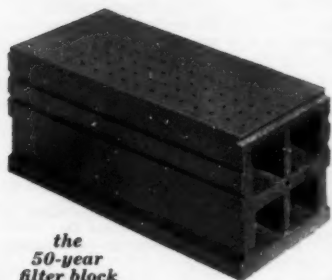
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Morgan Steel Products, Inc.
Pittsburgh-Des Moines Steel Co.

Tapping-Drilling Machines:

Hays Mfg. Co.
Mueller Co.
A. P. Smith Mfg. Co.

Tapping Machines, Corp.:

Hays Mfg. Co.
Mueller Co.

Taste and Odor Removal:

B-I-F Industries, Inc.—Builders
B-I-F Industries, Inc.—Proportion-
ers
General Filter Co.
Industrial Chemical Sales Div.
Permutit Co.
Wallace & Tiernan Inc.

Turbidimetric Apparatus (For Turbidity and Sulfate Determinations):

Wallace & Tiernan Inc.

Turbines, Steam:

Allis-Chalmers Mfg. Co.
DeLaval Steam Turbine Co.

Valve Boxes:

James B. Clow & Sons
Ford Meter Box Co.
M & H Valve & Fittings Co.
Mueller Co.
Rockwell Mfg. Co.
A. P. Smith Mfg. Co.
Trinity Valley Iron & Steel Co.
R. D. Wood Co.

Valve-Inserting Machines:

Mueller Co.
A. P. Smith Mfg. Co.

Valves, Altitude:

Allis-Chalmers Mfg. Co., Hydraulic
Div.
Golden-Anderson Valve Specialty Co.
Ross Valve Mfg. Co., Inc.

Valves, Butterfly, Check, Flap,

Foot, Hose, Mud and Plug:
Allis-Chalmers Mfg. Co., Hydraulic
Div.

B-I-F Industries, Inc.—Builders

James B. Clow & Sons
DeZurik Corp.
Kennedy Valve Mfg. Co.
M & H Valve & Fittings Co.
Mueller Co.
Pelton Div., Baldwin-Lima-Hamil-
ton

Henry Pratt Co.

Rockwell Mfg. Co.

R. D. Wood Co.

Valves, Detector Check:

Hersey-Sparling Meter Co.

Valves, Electrically Operated:

Allis-Chalmers Mfg. Co., Hydraulic
Div.

B-I-F Industries, Inc.—Builders

James B. Clow & Sons
Darling Valve & Mfg. Co.
Golden-Anderson Valve Specialty Co.
Kennedy Valve Mfg. Co.
M & H Valve & Fittings Co.
Mueller Co.
Henry Pratt Co.
Rockwell Mfg. Co.
A. P. Smith Mfg. Co.

Valves, Float:

James B. Clow & Sons
Golden-Anderson Valve Specialty Co.
Henry Pratt Co.
Rockwell Mfg. Co.
Ross Valve Mfg. Co., Inc.

Valves, Gate:

James B. Clow & Sons
Darling Valve & Mfg. Co.
Dresser Mfg. Div.
Kennedy Valve Mfg. Co.
M & H Valve & Fittings Co.
Mueller Co.
A. P. Smith Mfg. Co.
R. D. Wood Co.

Valves, Hydraulically Operated:

Allis-Chalmers Mfg. Co., Hydraulic
Div.

B-I-F Industries, Inc.—Builders

James B. Clow & Sons
Darling Valve & Mfg. Co.
DeZurik Corp.
Golden-Anderson Valve Specialty Co.
Kennedy Valve Mfg. Co.
F. B. Leopold Co.
M & H Valve & Fittings Co.
Mueller Co.
Pelton Div., Baldwin-Lima-Hamil-
ton

Henry Pratt Co.

Rockwell Mfg. Co.

A. P. Smith Mfg. Co.

R. D. Wood Co.

Valves, Large Diameter:

Allis-Chalmers Mfg. Co., Hydraulic
Div.

James B. Clow & Sons
Darling Valve & Mfg. Co.
Golden-Anderson Valve Specialty Co.
Kennedy Valve Mfg. Co.
M & H Valve & Fittings Co.
Mueller Co.
Pelton Div., Baldwin-Lima-Hamil-
ton
Henry Pratt Co.
Rockwell Mfg. Co.
A. P. Smith Mfg. Co.
R. D. Wood Co.

Valves, Regulating:

Allis-Chalmers Mfg. Co., Hydraulic
Div.

DeZurik Corp.
Golden-Anderson Valve Specialty Co.
Mueller Co.
Henry Pratt Co.
Rockwell Mfg. Co.
Ross Valve Mfg. Co.

Valves, Swing Check:

James B. Clow & Sons
Darling Valve & Mfg. Co.
Golden-Anderson Valve Specialty Co.
M & H Valve & Fittings Co.
Mueller Co.
Rockwell Mfg. Co.
A. P. Smith Mfg. Co.
R. D. Wood Co.

Venturi Tubes:

B-I-F Industries, Inc.—Builders
Rockwell Mfg. Co.
Simplex Valve & Meter Co.

Waterproofing:

Inertel Co., Inc.
Koppers Co., Inc.
Plastics & Coal Chemicals Div.,
Allied Chemical Corp.

Water Softening Plants; see Softeners**Water Supply Contractors:**

Layne & Bowler, Inc.

Water Testing Apparatus:

LaMotte Chem. Products Co.
Wallace & Tiernan Inc.

Water Treatment Plants:

American Well Works
Chain Belt Co.
Chicago Bridge & Iron Co.
Dorr-Oliver Inc.
Eimco Corp., The
General Filter Co.
Hungerford & Terry, Inc.
Inflico Inc.
Permutit Co.
Pittsburgh-Des Moines Steel Co.
Roberts Filter Mfg. Co.
Walker Process Equipment, Inc.
Wallace & Tiernan Inc.

Well Drilling Contractors:

Layne & Bowler, Inc.

Well Reconditioning and**Formation Testing:**

Halliburton Co.

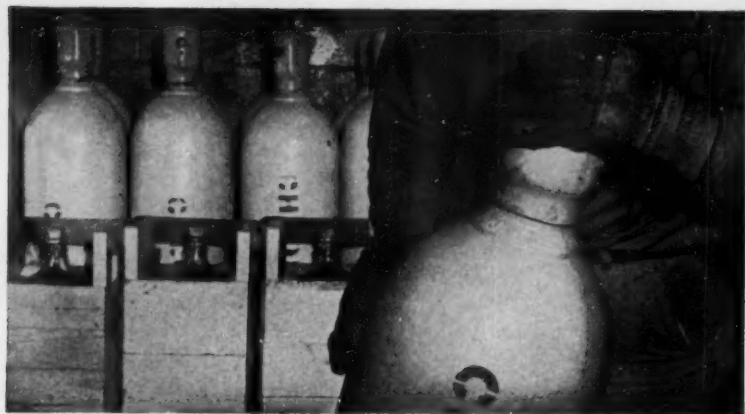
Layne & Bowler, Inc.

Wrenches, Ratchet:

Dresser Mfg. Div.

Zeolite: see Ion Exchange**Materials**

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